ONTARIO THRUWAY
MONTEZUMA SWAMP
SPOIL BANK SLIDE
CLYDE RIVER RELOCATION

Strength of spoil bank

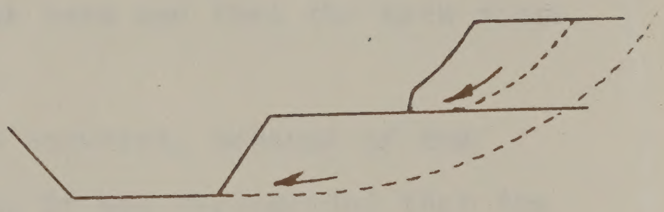
$\gamma = 85$ #/FT³ SATURATED

$H = 12'$

$i = 18.5°$

$\frac{c}{\gamma H} = 0.147 \quad (\phi = 0)$

$c = 0.147 \times 85 \times 12 = 150$ #/FT²



At the time of the slide, there were slides in both the spoil bank and in the marl foundation. Strength values of $\phi = 12.5°$ $c = 300$ #/FT² for the marl were obtained from the lab. tests for the Thruway crossing Montezuma Swamp. Strength valves of the spoil bank were obtained from Taylor's curves by the above computations to be c = 150#/FT², $\phi = 0°$.

In the vicinity of Sta. N 667 to N679 on the northernly side of the new Clyde River Channel, a rotational slide occurred involving approximately 7500 cu. yds. of material.

The section being excavated was a 55 ft. bottom width with one on three side slopes, the cut being approximately 22 ft. in depth. A 30 ft. berm was shown on the plans between the top of the slope and the area in which the spoil was being deposited. The height of spoil bank scaled 5' as per a verbal recommendation but was not dimensioned on plans.

On the basis of a field inspection and an analysis of the forces involved in the slide as it occurred, in which average strength values were used for the materials involved as indicated by laboratory data an similar materials, it appears that the cross section as designed and indicated on the contract plans was adequate if the height of the spoil bank behind the 30ft. berm did not exceed a height of 8 ft. It was recommended that no spoil material be placed on the berm and that the back slope of the berm be held at 1 on 3.

In that area where the slide occurred, because of the remolding of the subsurface soils, it was recommended that the height of spoil area in back of the 30 ft. berm not exceed 5 ft.

CATSKILL THRUWAY
DELAWARE AVENUE SLIDE

The following are recommendations made September 1951, for the reconstruction of the slide at Stations 1974 to 1977 of the above project:

1) extend the berm an additional 90 feet, for a total of 140 feet, continuing the original surface on a slope of 1 an 50 to the shoulder edge. The end slope should be 1 on 2 1/2 to permit use of unsuitable materials from clay cuts. The berm should merge with the surrounding ground with a smooth transition. The length of berm has been made to cover the toe of the mud wave developed within the slide area.

2) an inspection of the 36 inch reinforced concrete pipe has shown that there is a definite shear break and half-diameter displacement approximately under the left embankment shoulder. Also, the 4 joints immediately upstream from the shear displacement showed minor vertical faulting and horizontal displacement in the order of 6 inches between joints. The pipe from the downstream end to the failure is to be excavated and relaid and the pipe extended 110 ft. to a new headwall to coincide with the end slope of the new berm.

The berm construction should precede or proceed concurrently with the new pipe excavation to provide stability at all times.

1) Strength of fill

Assumed for original C = 1000 PSF, = 0°, = 130 PCF

Laboratory results from block samples of the compacted silt and clay fill gave C = 0 PSF and = 28° above an apparent preconsolidation pressure of approximately 5000 psf. with a shearing stress of approx. 2700 psf at lower confining pressures.

2) Strength of berm

The berm was considered to have no strength. Since the berm is in tension during failure, this assumption is reasonable.

3) Strength of subsoil

Assumed C = 600 PSF = 10°, = 55 PCF Sub.

The undisturbed samples disclosed the existance of an old shear slide. Along the sliding surface, C = 450 PSF and = 0° to 10° should be used.

The original analysis provided for a 1 on 2 slope. A 1 on 2 1/2 slope was provided, thus increasing the stability slightly.

CATSKILL THRUWAY
DELAWARE AVENUE SLIDE

The following are recommendations made September 1951, for the reconstruction of the slide at Stations 1974 to 1977 of the above project:

1) extend the berm an additional 90 feet, for a total of 140 feet, continuing the original surface on a slope of 1 on 50 to the shoulder edge. The end slope should be 1 on 2 1/2 to permit use of unsuitable materials from clay cuts. The berm should merge with the surrounding ground with a smooth transition. The length of berm has been made to cover the toe of the mud wave developed within the slide area.

2) an inspection of the 38 inch reinforced concrete pipe has shown that there is a definite shear break and half-diameter displacement approximately under the left embankment shoulder. Also, the 4 joints immediately upstream from the shear displacement showed minor vertical faulting and horizontal displacement in the order of 8 inches between joints. The pipe from the downstream end to the failure is to be excavated and relaid and the pipe extended 110 ft. to a new head-wall to coincide with the end slope of the new berm.

The berm construction should precede or proceed concurrently with the new pipe excavation to provide stability at alltimes.

1) Strength of Fill

Assumed for original C = 1000 PSF, = 0°, = 130 PCF

Laboratory results from block samples of the compacted silt and clay fill gave C = 0 PSF and = 28° above an apparent pre-consolidation pressure of approximately 5000 psf. with a shearing stress of approx. 2700 psf at lower confining pressures.

2) Strength of berm

The berm was considered to have no strength. Since the berm is in tension during failure, this assumption is reasonable.

3) Strength of subsoil

Assumed C = 600 PSF = 10°, = 55 PCF Sub.

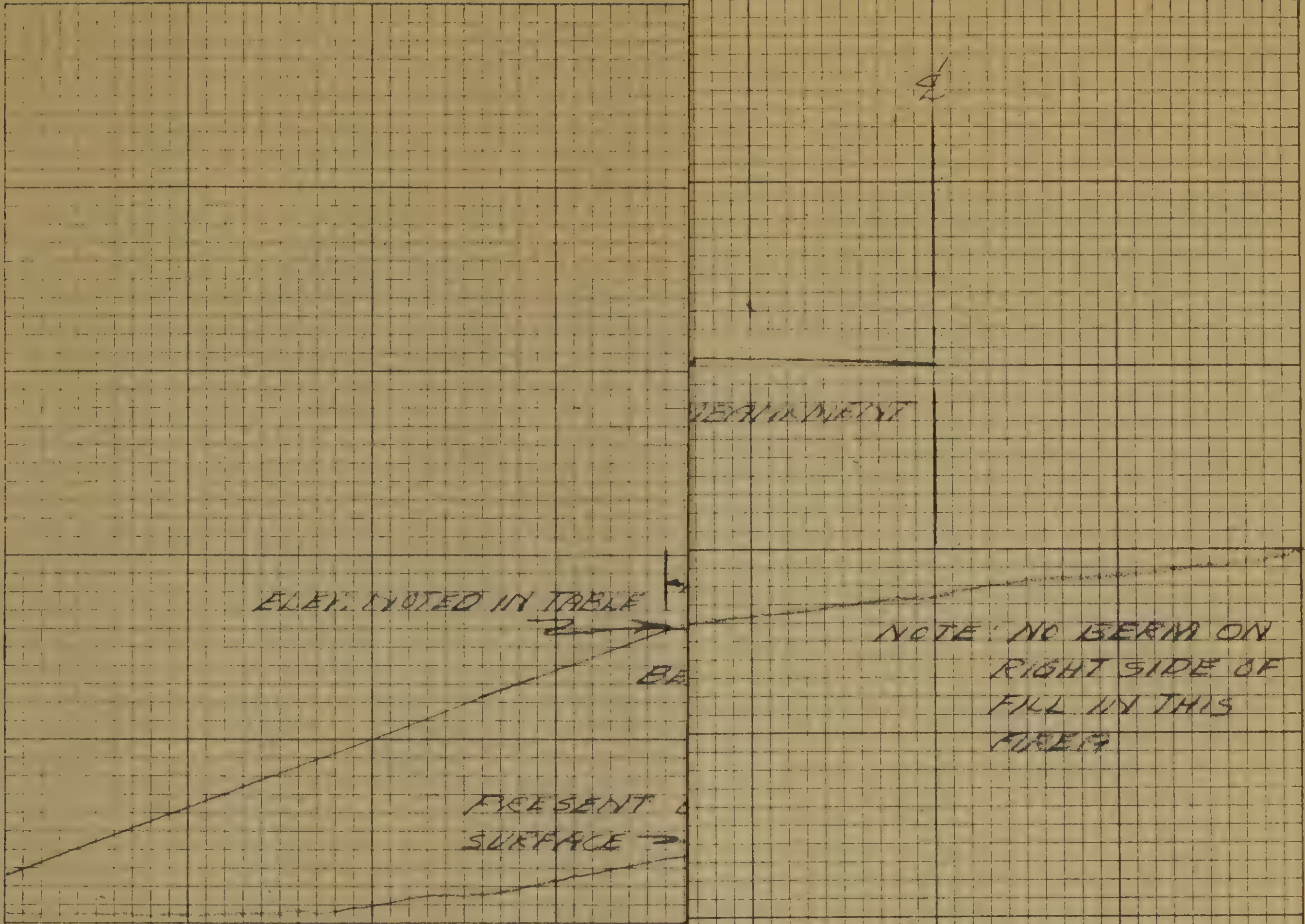
The undisturbed samples disclosed the existence of an old shear slide. Along the sliding surface, C = 450 PSF and = 0° to 10° should be used.

The original analysis provided for a 1 on 2 slope. A 1 on 2 1/2 slope was provided, thus increasing the stability slightly.

All cross-sections taken subsequent to the slide disclosed the berms lacked an average of 2 ft in height and 5 ft. in width. The slide, with its center over the berm, disallows the possibility of the slide motion causing an apparent decrease in berm size.

A widening of the fill to allow for the addition of an acceleration lane caused a worse condition than anticipated. The effective height of fill under the shoulder is increased by moving the shoulder outward over a ground surface that is sloping downward from the center line.

The original analysis assumed a constant unit weight, but the plans allowed for unsuitable material weighing approximately 110 PCF to be used in the berm. Therefore, in every 8 ft. of berm height, 1 ft. of effective weight is lost.



BERM SCHEDULE

STATION	BERM LEFT SIDE OF FILL (EL.)	WIDT
1974+00	NONE	NONE
1974+50	158	25
1975+00	158	[illegible]
1975+50	158	50
1976+00	158	50
1976+50	158	25
1977+00	NONE	NONE

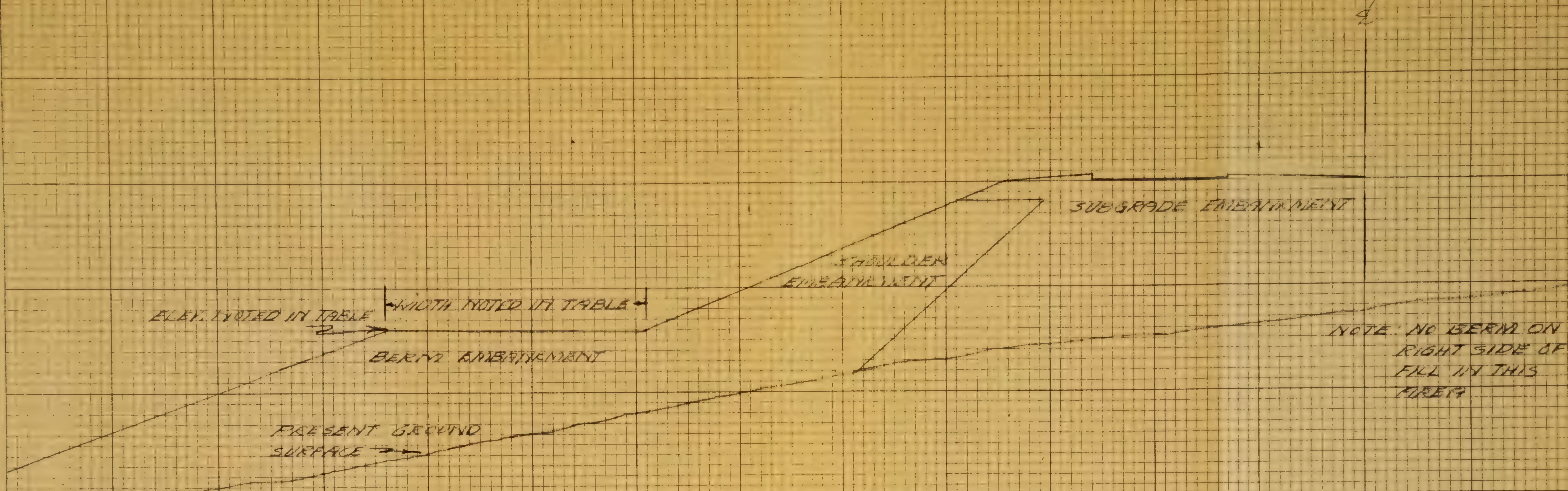
NOTE: AT THE ENDS OF AL
BERMS A TRANSITION
SHALL BE MADE TO
CONFORM WITH EXIS
GROUND CONDITION

DEPARTMENT OF PUBLIC WORKS
DIVISION OF CONSTRUCTION
BUREAU OF SOIL MECHANICS

CATSKILL THRUWAY
CORNING HILL - WESTERN AVE.
TYPICAL SECTION OF FILL WITH
BERM - STA. 1974+00 - 1977+00

APPROVED

DISTRICT 1
COUNTY ALBANY
DRAWING 1 of 8



STA. 1974+00 - 1977+00

BERM SCHEDULE

STATION	BERM LEFT SIDE OF FILL (EL.)	WIDTH
1974+00	NONE	NONE
1974+50	158	[illegible]
1975+00	158	[illegible]
1975+50	158	50'
1976+00	158	50'
1976+50	158	[illegible]
1977+00	NONE	NONE

NOTE: AT THE ENDS OF ALL BERMS A TRANSITION SHALL BE MADE TO CONFORM WITH EXISTING GROUND CONDITIONS

SCALE 1"=20'

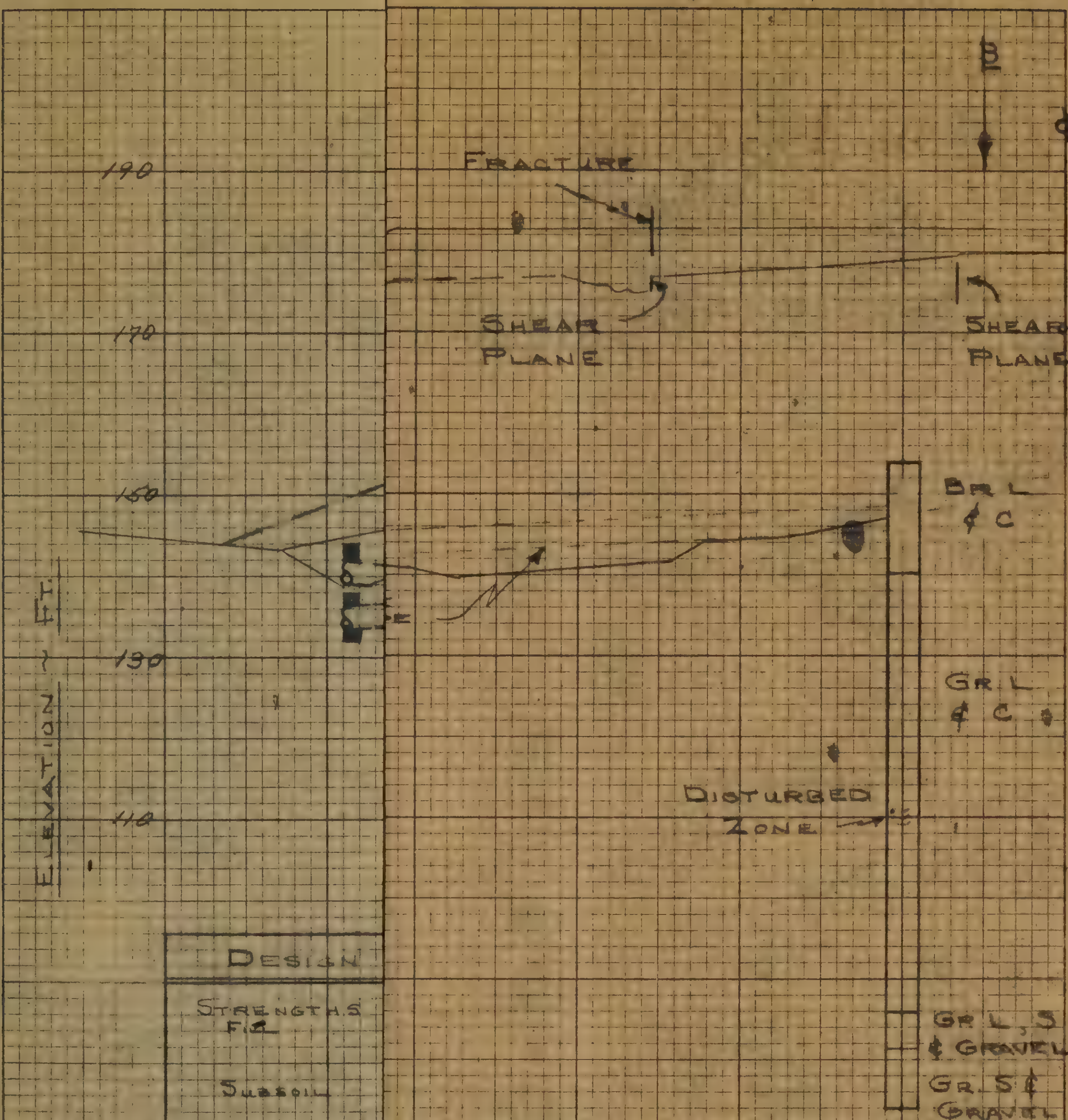
CATSKILL THRUWAY
CORNING HILL - WESTERN AVE.
TYPICAL SECTION OF FILL WITH BERM - STA. 1974+00 - 1977+00

APPROVED

DISTRICT NO. 1
ALBANY

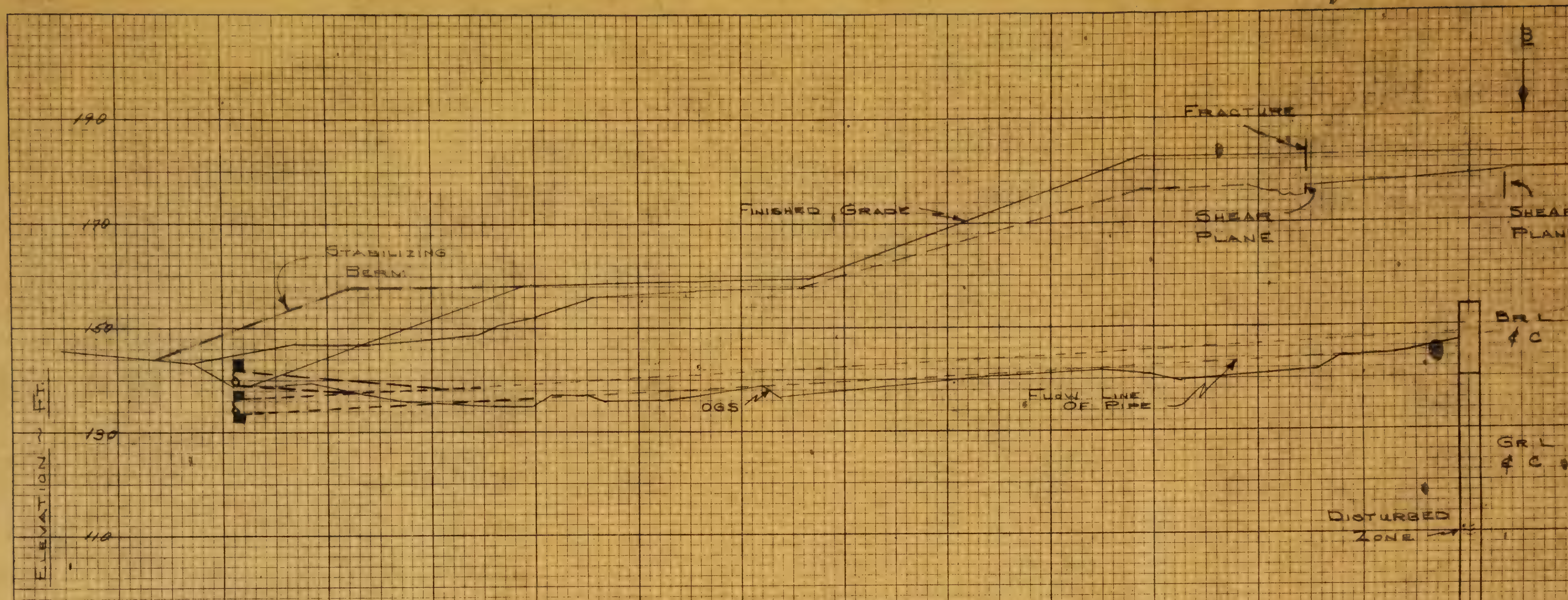
DRAWING NO. 1

KEUFFEL & ESSER CO., N. Y. NO. 359T-5LG
10 X 10 to the inch
MADE IN U.S.A.



Design
Strengths F.F.
Subsoil
Berm
No Tension
No Stren
No Reduc
Caused
Used 1
Cross- Se
Is Aver
In Slide
6 Circles
And Low
Berm Comp
To Gra
Fill Of S
Fill & Berm

Catskill Thruway
Delaware Ave. Slide
Sta. 1975±



DESIGN CONDITIONS	CONDITIONS AT FAILURE
STRENGTHS FILL $c = 1000$ #/FT² $\phi = 0°$ $\gamma = 130$ #/FT³	STRENGTHS FILL $c = 2700$ #/FT² $\phi = 25°$ $\gamma = 127$ #/FT³
SUBSOIL $c = 400$ #/FT² $\phi = 10°$ $\gamma = 55$ #/FT³ SUBMERGED	
BERM $\gamma = 110$ #/FT³	
NO TENSION CRACKS CONSIDERED. NO STRENGTH IN THE BERM. NO REDUCTION IN STRENGTH CAUSED BY FORMER SLIDE	EVIDENCE OF $c = 300$ TO 600 #/FT² $\phi = 5°$ TO $10°$
USED 1 ON 2 SLOPES	SLOPES WERE 1 ON 2½
CROSS-SECTION AT STA 1975+50 IS AVERAGE OF CONDITIONS IN SLIDE 6 CIRCLES GAVE CRITICAL CIRCLE AND LOWEST SAFETY FACTOR	
BERM COMPLETED BEFORE FILL WAS TO GRADE	BERM NOT BROUGHT TO GRADE OR WIDTH
FILL OF STANDARD DIMENSIONS USED FILL & BERM A CONSTANT 130 #/FT³	110 #/FT³ UNDER SHOULDER AND BERM

DRILL HOLE
AT STA. 1975+93

CATSKILL THRUWAY
DELAWARE AVE. SLIDE
STA. 1975±

NEW BERLIN - SOUTH NEW BERLIN SLIDE

The soil profile, in general, consists of a layer of silt and clay which extends to a depth of 40 feet under the slide area and decreases in thickness to 0 feet approximately 200 feet left, or on the hillside, of the center line. A continuation of the sand and gravel terrace which is located beyond 200 feet left of the center line extends beneath the silt and clay, and lies directly on shale bedrock or compact till which in places overlies the bedrock. This sand layer which is under artesian pressure decreases in thickness to approximately 20 feet under the Unadilla River.

Following are methods of stabilizing the slide:

(1) Drainage system

Basically, this is the best approach since it remedies the cause of instability by reducing the artesian pressure. The installation of drainage wells would affect a permanent cure, but would be more costly than a berm.

(2a) Counterweight berm

This method would afford at least a temporary stability for the sliding area by creating a more favorable distribution of stresses within the sliding mass. Should the artesian pressures increase to some point above the present pressure, the slide may move again. The portion of the berm adjacent to the river may in itself become unstable and slide in the river.

(b) Relocating the Unadilla River and using a larger berm.

This method could affect a degree of stability comparable to or better than the drainage system. There would be the additional expense of R.O.W. and channel excavation using this method.

[illegible]

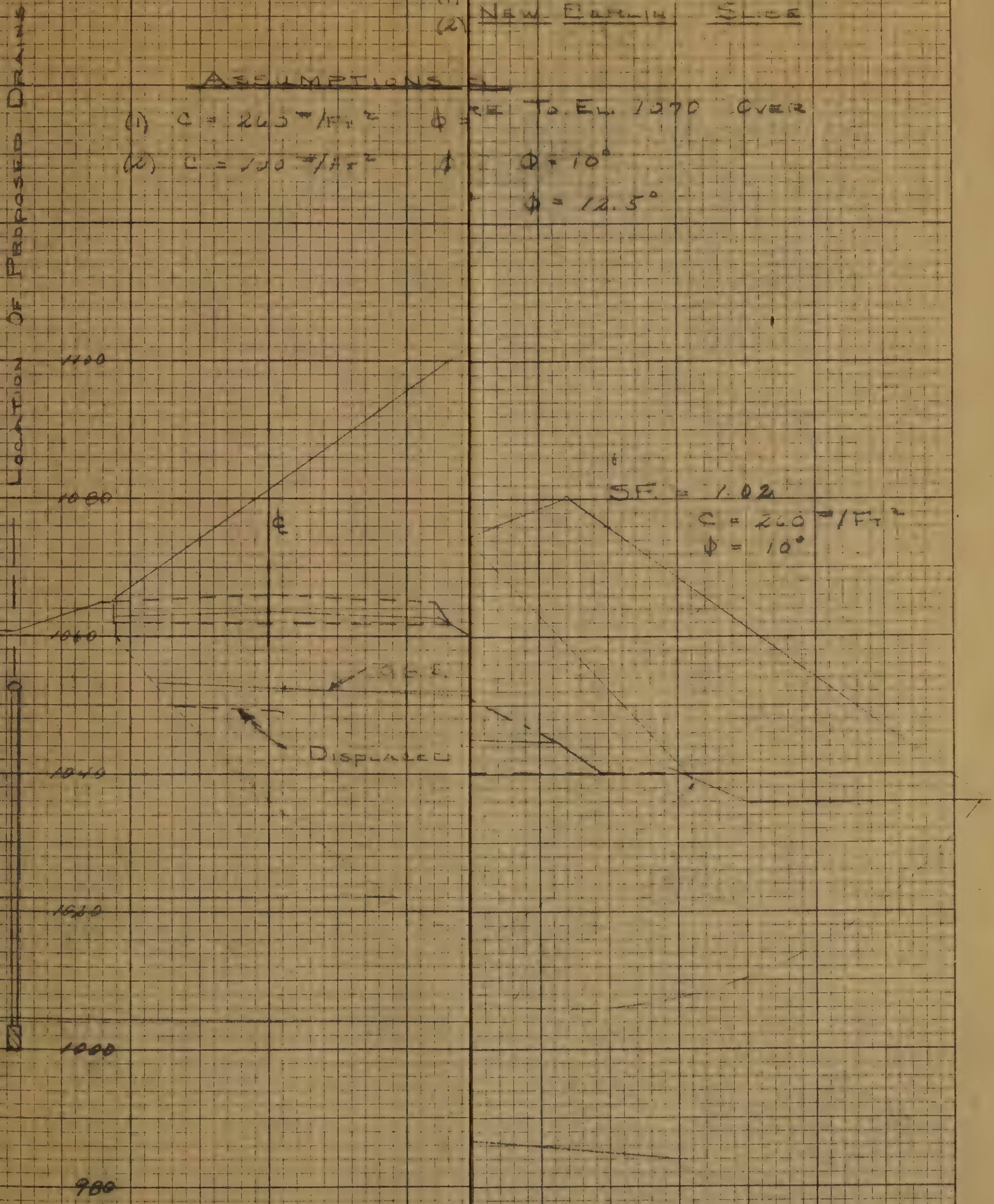
(1) New Berlin - South
(2) New Berlin Slide

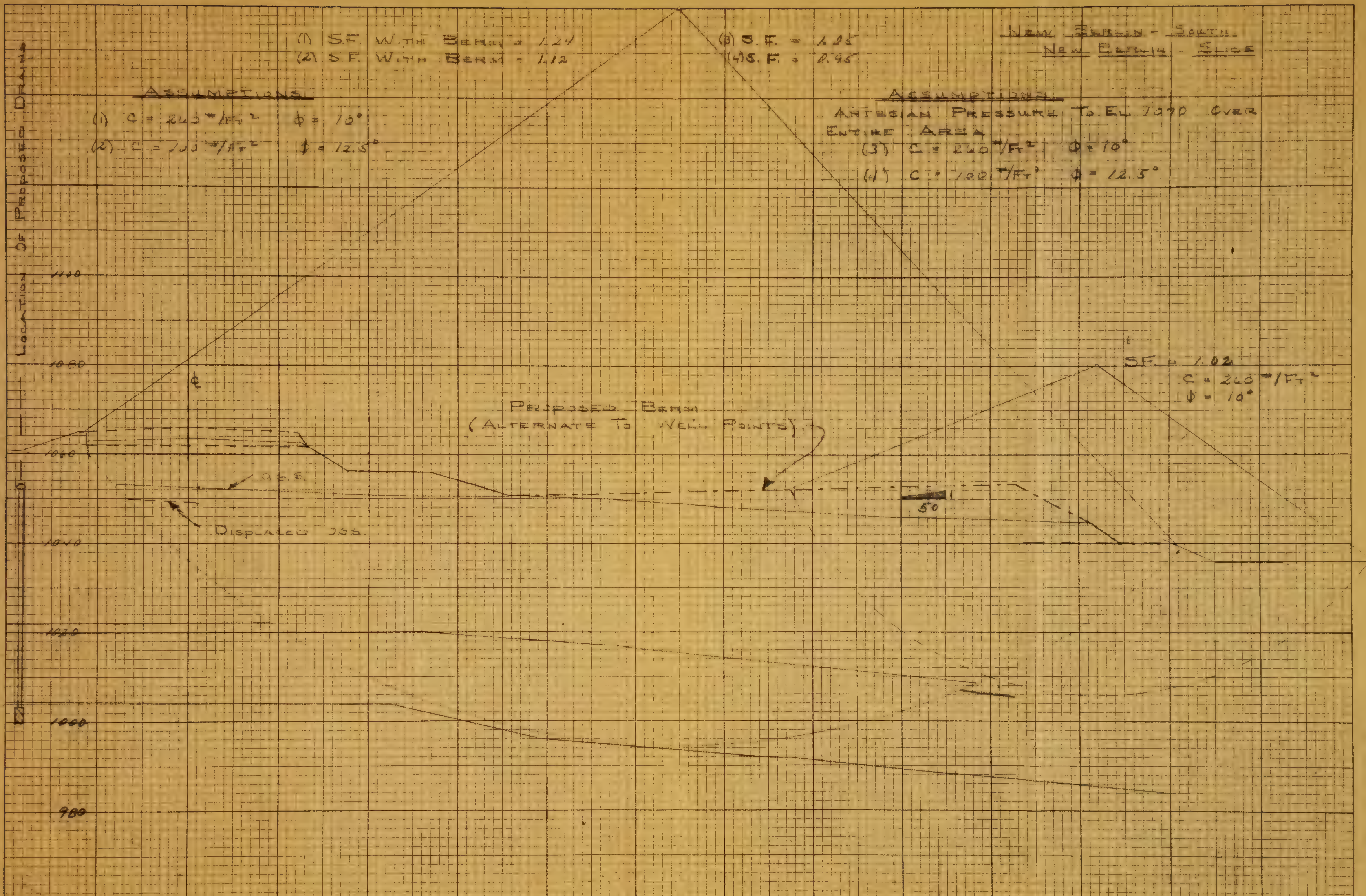
Assumptions

(1) $C = 200\ \#/Ft^2$ $\Phi =$ [illegible] To El. 1070 Over

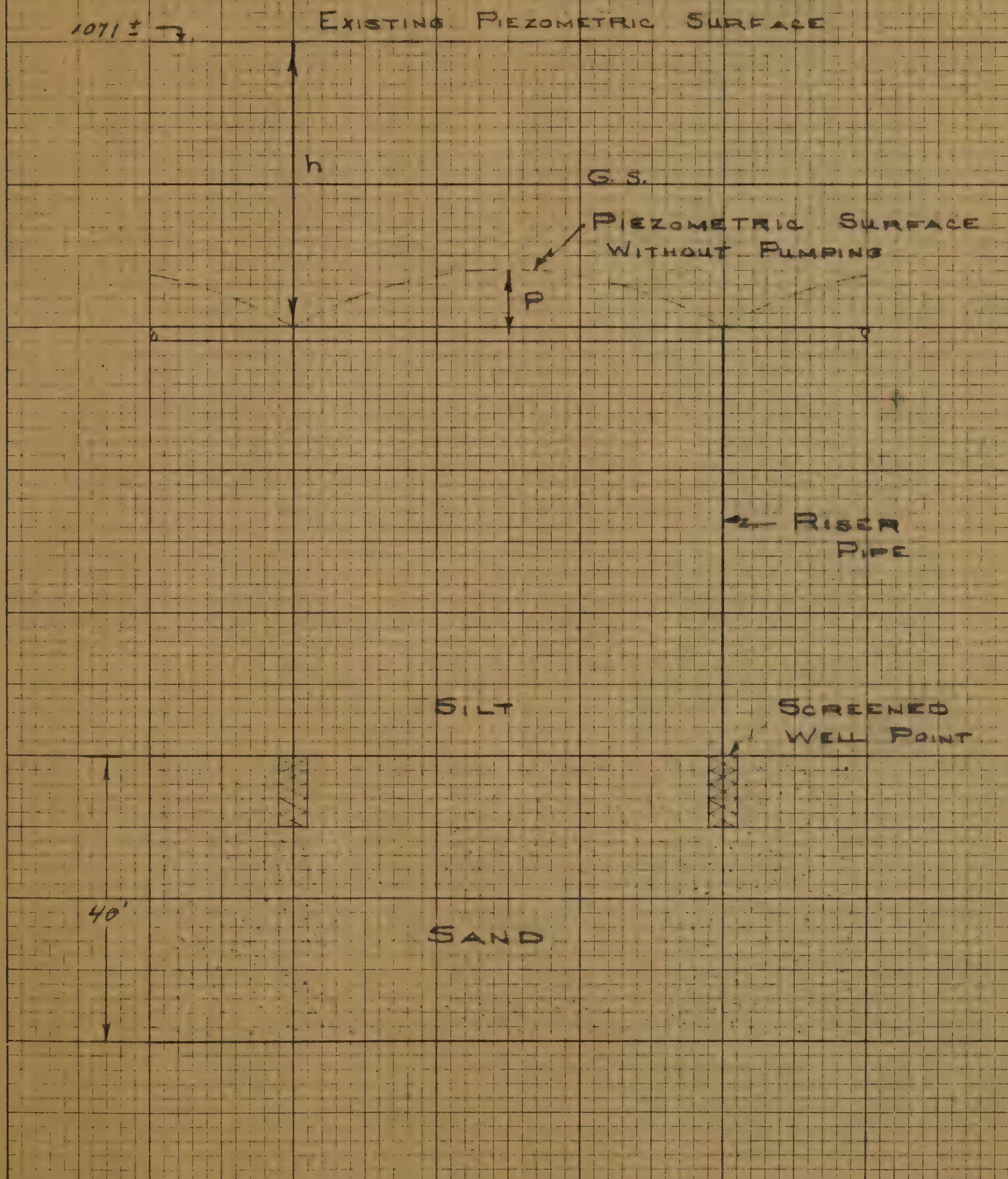
(2) $C = 100\ \#/Ft^2$ Φ $\Phi = 10°$

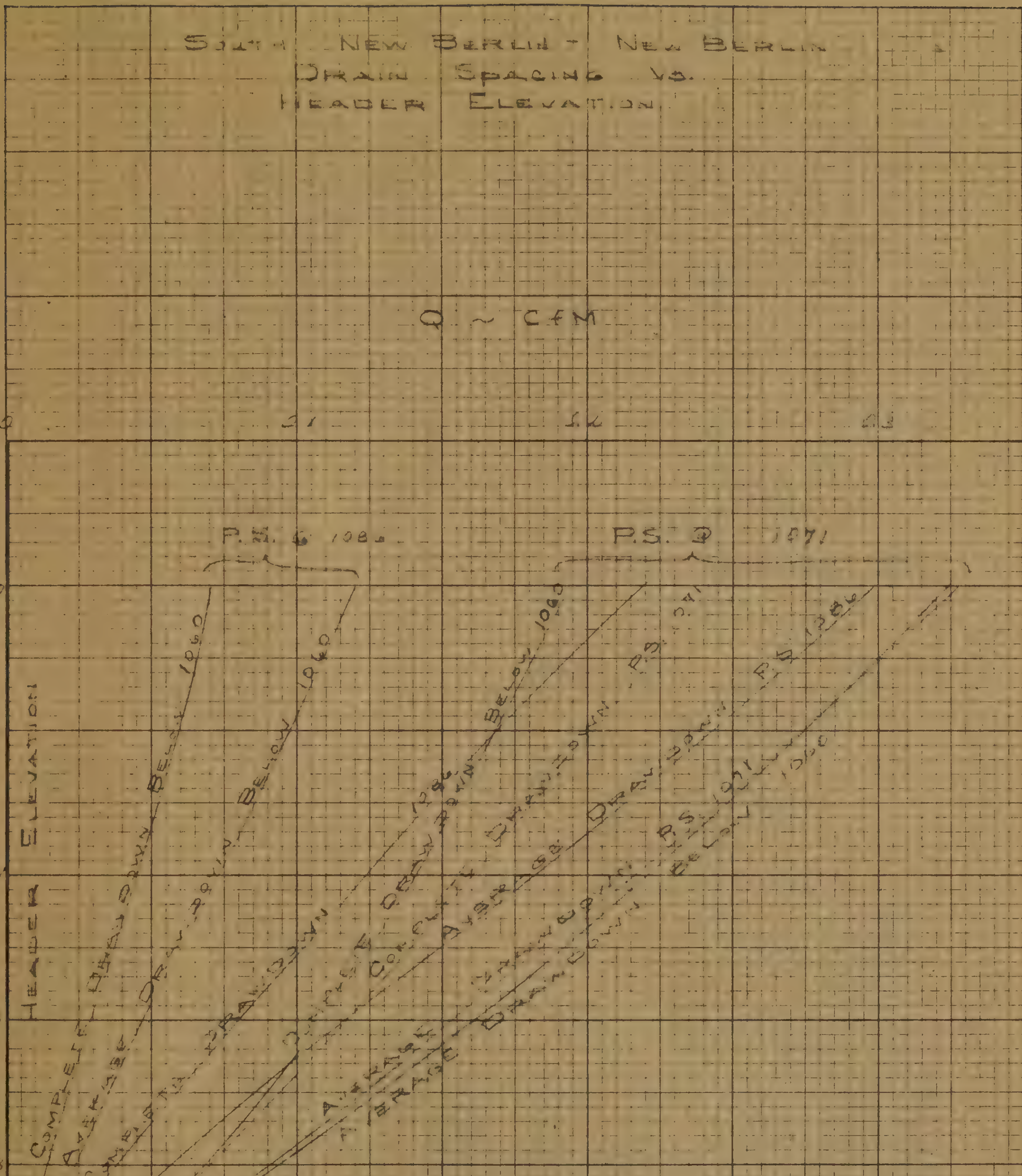
$\Phi = 12.5°$



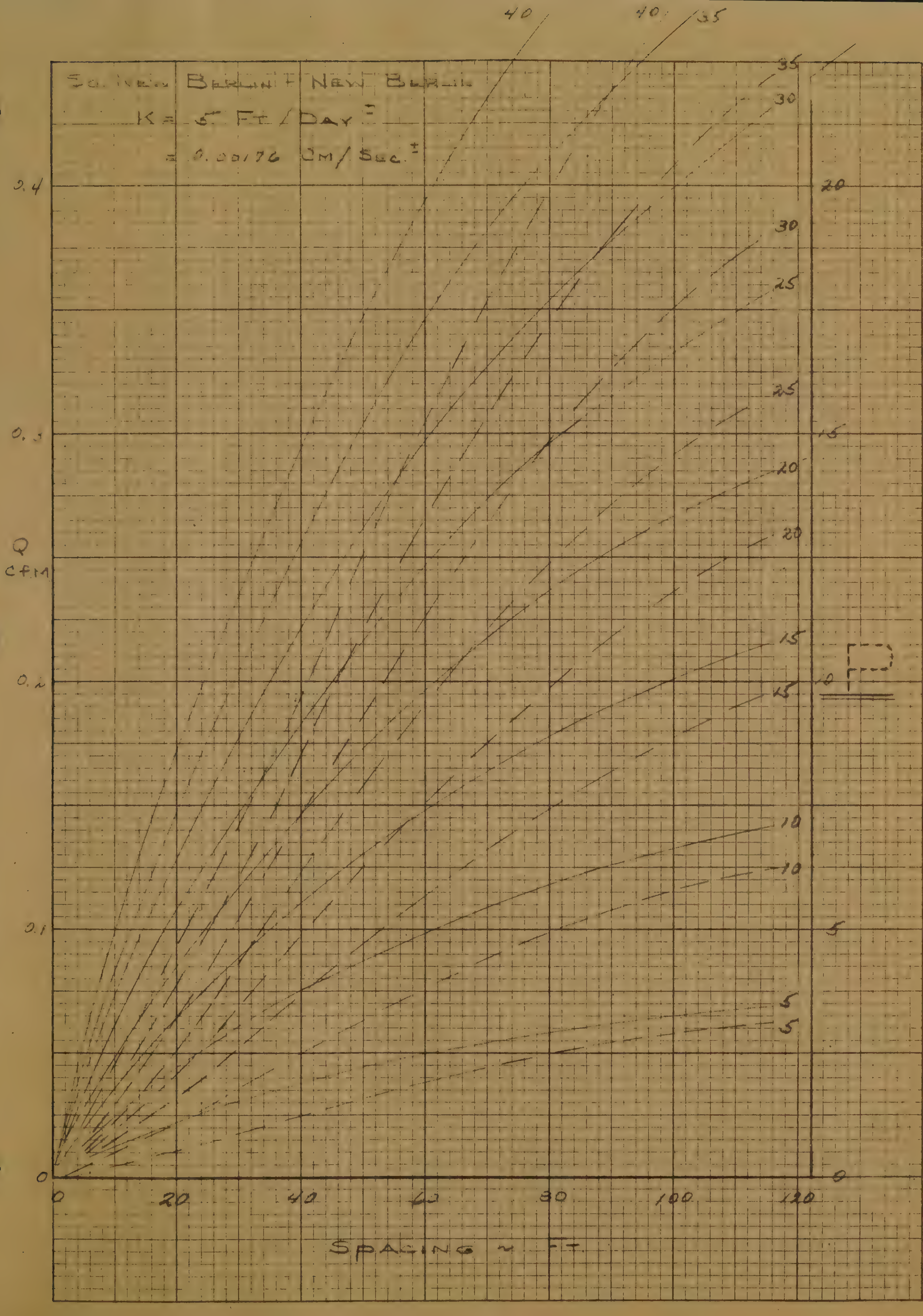

New Berlin - South
New Berlin Slide
(1) S.F. With Berm = 1.24
(2) S.F. With Berm = 1.12
(3) S.F. = 1.25
(4) S.F. = 0.95
Assumptions
(1) C = 240 #/Ft² Φ = 10°
(2) C = 100 #/Ft² Φ = 12.5°
Assumptions
Artesian Pressure To El. 1070 Over
Entire Area
(3) C = 240 #/Ft² Φ = 10°
(4) C = 100 #/Ft² Φ = 12.5°
S.F. = 1.02
C = 240 #/Ft²
Φ = 10°
Location Of Proposed Drains
1100
1080
1060
1040
1020
1000
980
℄
Proposed Berm
(Alternate To Well Points)
O.G.S.
Displaced O.G.S.
50

South New Berlin + New Berlin Slide
(Not To Scale)

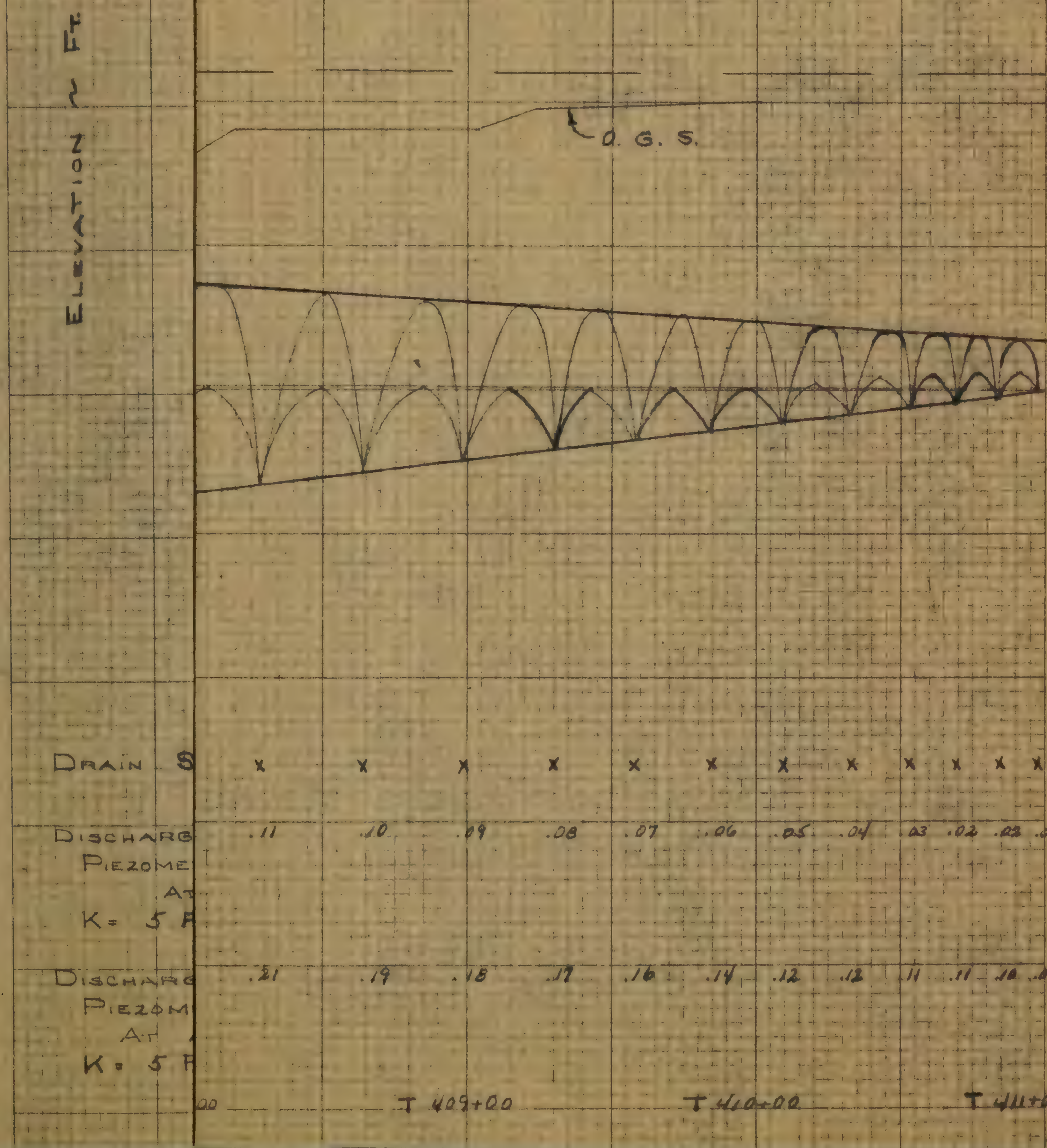


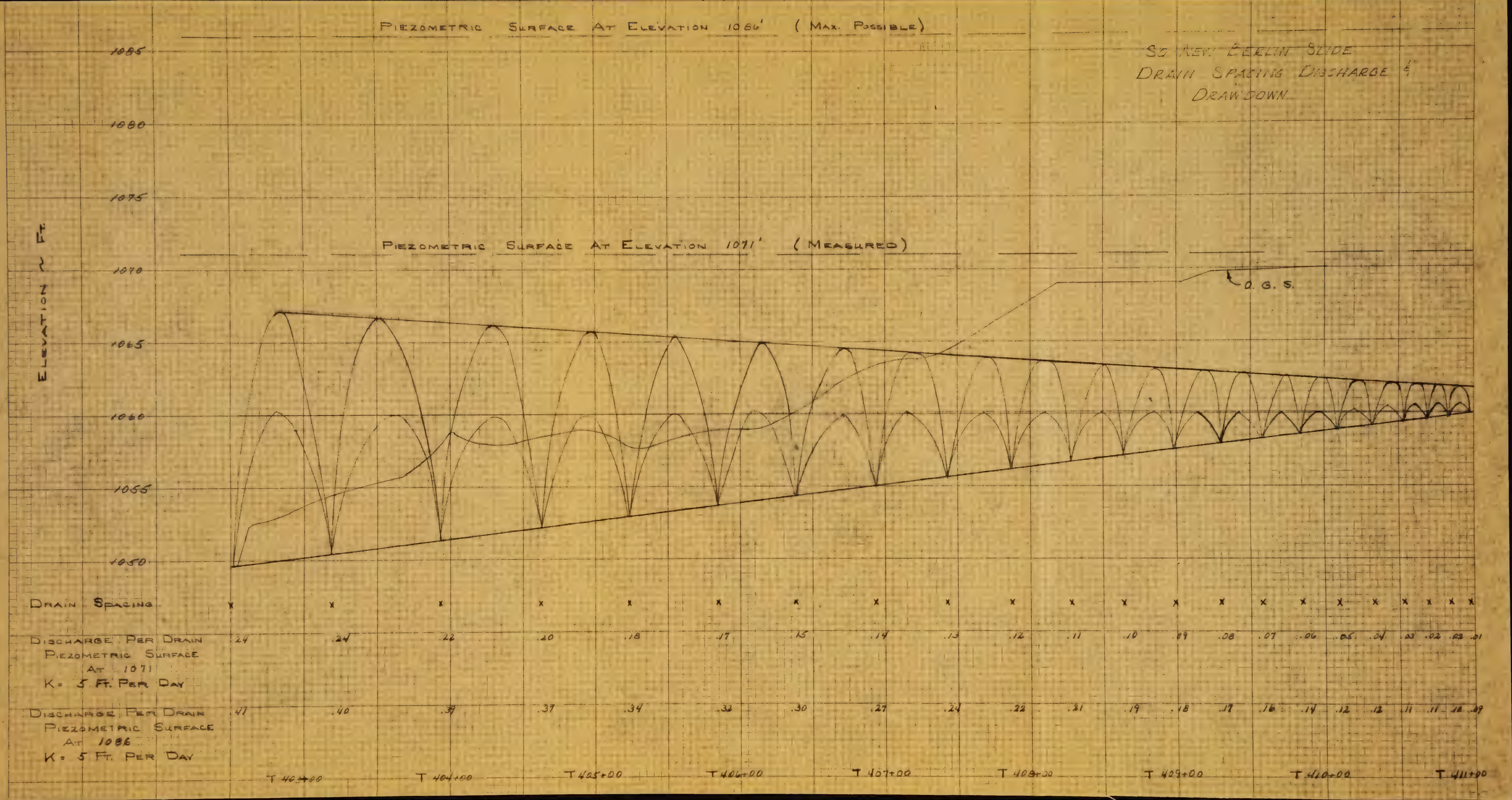


South New Berlin - New Berlin
Drain Spacing vs.
Header Elevation
Q ~ CFM
P.S. 6 1086
P.S. 3 1071
Header Elevation
1050
1052
1054
1056
1058
Drain Spacing ~ Ft


So. New Berlin & New Berlin
K = 5 Ft/Day ±
= 0.00176 Cm/Sec ±
Q
cfm
0.4
0.3
0.2
0.1
0
0
20
40
60
80
100
120
Spacing ~ Ft.
40
40
35
35
30
30
25
25
20
20
15
15
10
10
5
5
20
15
10
5
0

SO. NEW BERLIN SLIDE
DRAIN SPASING DISSHARGE &
DRAWDOWN




Piezometric Surface At Elevation 1086' (Max. Possible)
So. New Berlin Slide
Drain Spacing Discharge &
Drawdown
Elevation ~ Ft.
1085
1080
1075
Piezometric Surface At Elevation 1071' (Measured)
1070
O. G. S.
1065
1060
1055
1050
Drain Spacing
Discharge Per Drain
Piezometric Surface
At 1071
K = 5 Ft. Per Day
.24 .24 .22 .20 .18 .17 .15 .14 .13 .12 .11 .10 .09 .08 .07 .06 .05 .04 .03 .02 .02 .01
Discharge Per Drain
Piezometric Surface
At 1086
K = 5 Ft. Per Day
.47 .40 .39 .37 .34 .32 .30 .27 .24 .22 .21 .19 .18 .17 .16 .14 .12 .12 .11 .11 .10 .09
T 403+00
T 404+00
T 405+00
T 406+00
T 407+00
T 408+00
T 409+00
T 410+00
T 411+00

Presented at the N.Y.S.H.E.A. Convention
Syracuse, N.Y. March 21, 1956
by M.N. Sinacori

EMBANKMENT FOUNDATION PROBLEMS IN SYRACUSE AREA

During the last few years, there have been many foundation problems in connection with highway design and construction in the Syracuse area, mostly centered around the Onondaga Lake perimeter. The general nature of the foundation conditions and development.

The fact that some of these areas have not been more fully developed industrially has reflected, to a large degree, the very poor foundation conditions existing. To me, it is a tribute to the Syracuse District Engineer and to his engineering organization for their foresight and boldness of decisions in locating their newly required major highways through these partly unoccupied areas, which are the only areas left, even though the foundation problems have been and are of major consideration.

Figure No. 1 shows a plan location of these main highways which either have been built, are under construction, or are in various stages of design. Going across the top of the slide is the New York State Thruway. This, of course, has been completed. The foundation problems encountered for this project are in the vicinity of the lake outlet and are shown shaded near the No. 1.

Coming around the lake in a counter-clockwise manner, we have the Lake Onondaga West Shore Development, the State Fair Grounds Park Area, and the State Fair Boulevard reaching well into the city. All of these are in various stages of construction, or just completed. The major areas of foundation problems are shown shaded and numbered (2), (3) and (4).

Coming to the southeast of the lake, there is Hiawatha Boulevard, and the Oswego Boulevard Extension. These are in various stages of design, and practically the entire length has critical foundation conditions. They are shwon by the shaded areas Nos. (5) and (6). The Oswego Boulevard Extension will eventually tie into the Lake Onondaga Parkway and into the recently completed Syracuse-Mattydale section, which also connects to the Thruway.

The present Onondaga Lake is a remnant of a more extensive shallow glacial lake. The soft foundation materials surrounding the limits of the present lake have accumulated during and following the last glacial period.

A typical foundation profile may consist of muck at the surface, underlain by soft marl, which merges into soft silt and clay below. Underlying these soft layers may be found sands and silts, then sands and gravels, and then compact glacial till above rock. The thicknesses of these layers may vary from place to place, and not all of them may be found in any one location.

Muck is an accumulation of vegetable matter, and is about the same as is found in other parts of the State. Marl is essentially calcium carbonate mixed with various proportions of impurities. It is soft, weak, and very compressible. The silt and clay is soft and compressible to a slightly lesser degree. The sands and silts are reasonably firm and form a good transition layer into the sands and gravels and tills below, which are firm and compact.

In addition to the natural causes of deposition, these foundation problems are further complicated by the man-made modifications. Nature is generally systematic in its work, even though its results may be hetrogeneous, but man's contribution may be quite variable and unpredictable. These include dump fills, remains of prior construction of different kinds, the presence of many utility lines, and sludge beds and layers of sludge found over a large part of the area. Sludge is a whitish waste material from the Solvay Company, and is generally very soft, greasy and fine-grained, and contains a very high moisture content.

To complicate the problems still further, there have been found hydrostatic pressures at some levels in the foundation, and also salt in solution in the ground water. These are of specific importance during design, as proper interpretation is essential for an accurate evaluation of settlement and strength relations.

I will attempt to describe briefly the foundation characteristics and design features for each of the major locations shown on this slide.

Figure No. 2 is a profile of the foundation conditions along the Thruway at the lake outlet. The soft material extends to a depth of approximately 70 feet, and consists of muck, marl and silt and clay. The fill height to grade is approximately 22 feet. Three variables of design were used, depending on the depth and nature of the soft foundation materials: (a) Sand drains were used for the westerly portion where the soft foundation material was the deepest; (b) Partial excavation of the muck was used in the central portion. The fill was built on the marl and silt and clay, and a surcharge was placed above final grade to hasten settlement; (c) Complete excavation to remove all soft material was used in the easterly portion. Typical cross-sections for each of these three methods of solution are shown in Figure No. 3. These are economically in balance. In the easterly portion, shown by Section C-C, it was cheaper to remove all of the soft material which was about 10 feet deep. In the mid-section, shown by Section B-B, the compressible layers were 20 to 30 feet thick, and partial excavation and the use of surcharge was more economical. Due to the great depth of soft material in the westerly portion, to some 70 feet, as shown by Section A-A, sand drains were used to obtain stability and settlement adjustment.

Figure No. 4 shows the amount and rate of settlement obtained in the foundation at one location within the sand drain area. The horizontal scale is a time scale and is common to both curves.

The vertical scale for the upper curve shows the height of fill reached at any time. The vertical scale for the lower chart is the settlement in feet. The solid curve shows the settlement computed for design before the project was built, and the dashed curve the settlement actually measured in the field during construction. As you can see, the total settlement obtained approached 9-1/2 feet.

Figure No. 5 shows a profile of the Lake Onondaga West Shore Development north of the sludge beds and the State Fair Grounds. Here, this road takes off from the top of the sludge beds and comes down across Nine-Mile Creek into the flats adjacent to the lake. The foundation consists of sludge, muck, marl, and silt and clay, and occasional man-made fills. During construction, the muck was removed and the fill placed either on the sludge or marl, with a surcharge added to hasten settlement. The worst foundation conditions were at Section A-A. At this location, the marl and silt and clay were deepest, stability was quite critical, and a shear failure was a distinct possibility. As it turned out, a shear failure did occur within the limits of the northbound lane during the placement of surcharge. The conditions were aggravated by the nearness of the creek which here runs parallel to the road. This failure has since been repaired.

Figure No. 6 shows a plan and typical section of the State Fair Grounds Parking Area and the northbound lanes for the West Shore Development, located on top of the lower sludge beds in front of the Fair Grounds. The section shows schematically the relative width and height of the lower sludge bed. This bed is approximately 800 feet wide and 3600 feet long and is approximately 25 feet high. The problem connected with constructing on these sludge beds was primarily to obtain shear stability and surface bearing capacity. Due to the soft nature of the material, getting onto the area was a problem in itself.

After considerable study, it was decided that a minimum of two feet of gravel over the area would permit use for parking. The specifications called for the first layer to be 18 inches thick, and that all construction equipment was confined to travel on top of the first layer. Subsequent layers were placed in 6-inch thickness and properly compacted. The overall shear stability problem was more critical in the areas where fills up to 10 feet in height were needed for leveling to the required grade elevations. Paradoxically, the greatest fill height was needed in the area where a slide had occurred some years ago, which caused a good part of this sludge material to flow and cover most of the area in front of the State Fair Grounds. The full height of material was placed according to schedule, and the fill has, to date, shown no signs of sliding.

Due to the nature of the sludge material and also due to the relatively soft foundation materials which exist under the sludge beds themselves, some settlement was expected over the entire area, as the result of placing this fill on top of the sludge beds. Since no economical method could be worked out that would eliminate settlement during construction, it was decided to provide for only bearing capacity and shear stability

and to take the settlement as it developed. During the first year after construction, settlements from 1 to 1.5 feet have been measured along the northbound arterial. The change of settlement, however, has been gradual and the road surface is still in fair riding quality. It is expected that, in the future, some grade adjustment may have to be made, especially in the Parking Area, to provide for positive drainage, but the cost of this correction will be quite low in comparison to the high cost of eliminating settlement during construction, if that were possible.

Figure No. 7 is a plan of the State Fair Boulevard connecting the sludge beds to Spencer Street, in the City of Syracuse, southwest of Onondaga Lake. This project is now under construction. You may see the existing State Fair Boulevard and its intersection with Hiawatha Boulevard at the center of the slide.

Figure No. 8 is a profile along the main line of the State Fair Boulevard, showing soft materials to a depth of approximately 100 feet, which consists of Solvay sludge, marl and soft silt and clay. The fill ranges to 27 feet in height. Due to the height of fill and the great depths of soft material, sand drains were used throughout the area to stabilize the foundation. These drains extended from the intersection with the present State Fair Boulevard to approximately 400 feet south of Hiawatha Boulevard. Although these fills have not yet been finished to final surcharge grade, foundation settlements up to six to seven feet have already been measured. South of this point, the original surface has been covered with some 20 feet of miscellaneous dump fill. Here, final grade was low enough to permit using heavy pneumatic-tired rolling to compress the surface materials and give a stable subgrade. The wolling was done with approximately 25 passes of a 50-ton pneumatic-tired roller. This is similar to the rolling done in other districts; such as, in Buffalo, Binghamton and Long Island. The average surface settlement obtained from the rolling was approximately two feet.

Figure No. 9 shows a profile and section of Hiawatha Boulevard in the vicinity of Oswego Boulevard. This project has been designed, but has not yet been let for construction. The fill is expected to be up to 23 feet high. The foundation material consists of dump cinder fill over muck and marl. The problem at this location was serious because of the presence of active utility lines and the limited R.O.W., requiring crowding of the existing Hiawatha Boulevard which has to be retained. The solution here involved complete excavation of the very soft materials, with restrictions placed on the width of excavation to protect some of the utility lines, and also permit relocating other lines prior to excavation.

Figure No. 10 shows the plan of the proposed Oswego Boulevard Extension from Hiawatha Boulevard to beyond Park Street, to connect with the Syracuse-Mattydale section already built. The present Park Street and its intersection with Hiawatha Boulevard are quite congested. The side ramp connecting Hiawatha Boulevard

to Park Street has already been built to alleviate traffic congestions at the intersection. The remainder of the project is under design. The problem at this location is complicated by the fact that the highway is crossing over the New York Central Railroad Spur, which itself is already on a fill approximately 11 feet high, requiring fills for the highway up to 40 feet in height.

Figure No. 11 is a profile of the main line from just north of Hiawatha Boulevard to Park Street, showing the high fills necessary to cross over the New York Central Railroad Spur. The existing Beartrap Creek flowing through the area is to be relocated next to the Railroad, to permit continuous embankment north of the Railroad fill. The soft foundation material is about 50 feet deep, and consists of shallow dump fills over marl and silt and clay, with some Solvay sludge south of the Railroad. The design is based on using sand drains in stabilizing the foundation. Due to the high fills involved, stabilizing side and end berms are also necessary to obtain shear stability. As you notice from the profile, the toes of the end berms were kept some distance away from the toes of the Railroad fill. This was done to insure that there be no settlement in the Railroad fill itself, as the result of constructing the approach fills on either side.

Figure No. 12 shows a continuation of the foundation profile along the main line north of Park Street. The existing Syracuse-Mattydale project, already built, starts just beyond the limit of the slide. Presently, this project is connected to Park Street with a temporary service road, the surface of which is just above the swamp level. To permit crossing over Park Street, the approach fill has to be raised to a height of approximately 22 feet. There will be involved the excavation of muck for widening, the spreading out of the existing temporary fill and leveling the area, placing a drainage blanket, and then driving the sand drains. These sand drains will be carried to Station 42/50. Beyond that point, the fill is shallow enough so that a surcharge height alone above final grade will be sufficient to give the necessary settlement adjustment during construction.

Figure No. 13 shows the profile and sections for the exit ramp over Onondaga Lake Parkway. The soft foundation soil is marl over silt and clay some 40 feet thick in the area away from the bridge, becoming shallower as the bridge location is approached. In the deeper areas, aided by the low fills, a surcharge is needed to obtain settlement adjustment. Within the limits of this Bridge structure, due to the shallower depth of soft material, complete excavation was considered more economical.

WEST SHORE DEVELOPMENT

On July 19, 1955 a slide occurred between Station 168 + 00 and 170 + 00. The earth mass was about 300 feet long, 48 feet wide, and nine feet high. Maximum settlement was about ten feet. Partial blocking of the creek occurred, but the creek soon washed out the sludge block and discharge flow returned to normal.

The soil profile at Station 170 + 00 consists of ten feet of muck, twenty feet of marl, forty feet of silt and clay overlying sand and silt. It is noteworthy that the failure occurred where the marl and silt and clay layers were thickest and deepest as shown on the accompanying profile.

On July 21 and 22, the contractor built a stabilizing berm. The berm was 300 feet long, twenty feet wide, and four feet thick. Reconstruction of the embankment was again attempted on July 22, 1955. It was to a height about one foot above finished pavement grade when it failed. The new fill dropped about 2½ feet toward the creek.

No further work was done in this area with the exception of providing drainage and filling in of small areas.

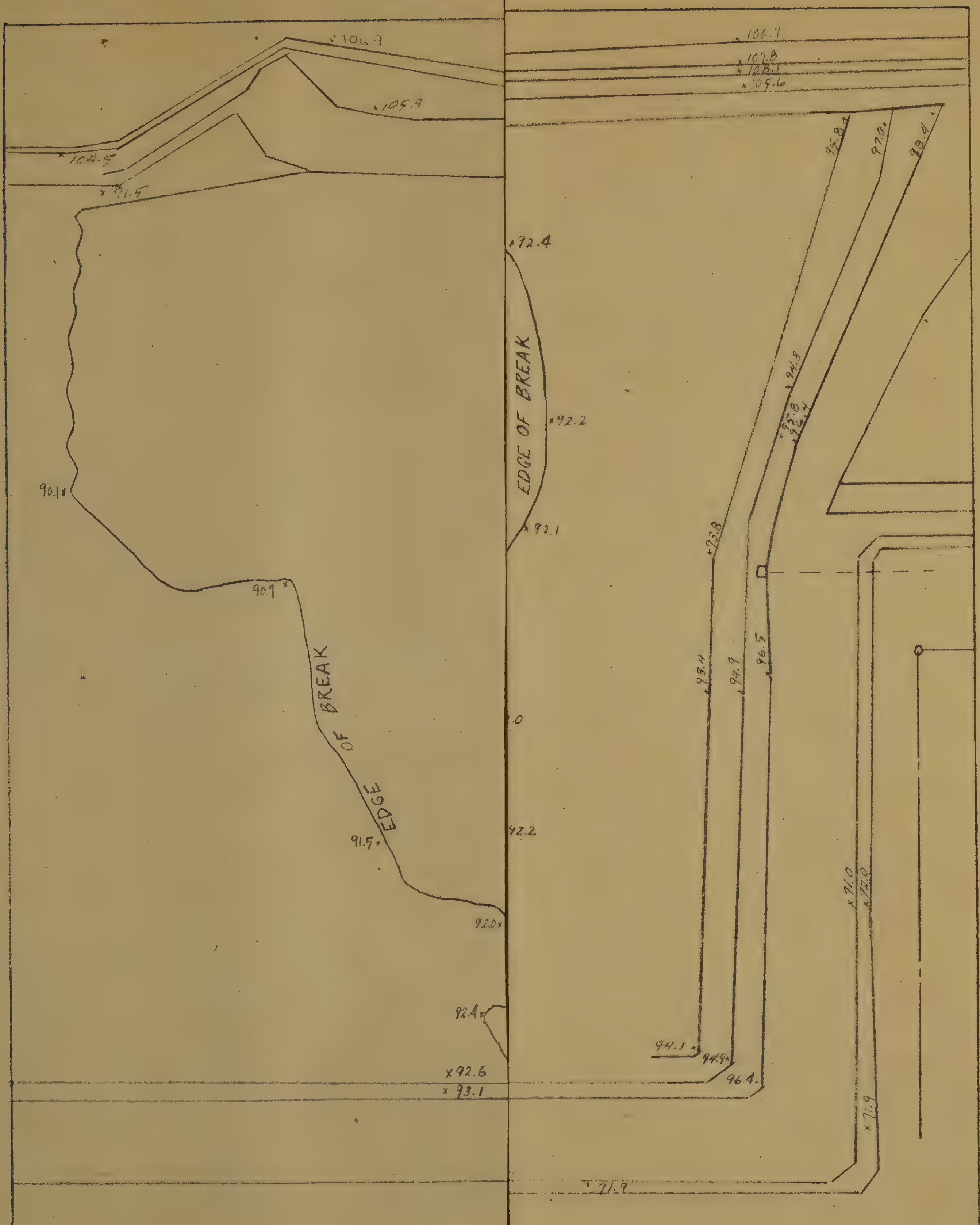
On July 22, 1955 it was concluded to attempt to rebuild the embankment in the slide area to one foot above finished grade at a time later in the year.

On September 15, 1955 it was suggested that fill could be placed in the slide area. A slow rate of construction, eight inches every three days, and utilization of light weight hauling equipment so as to minimize vibration and impact was recommended to reduce the tendency for further shearing displacement.

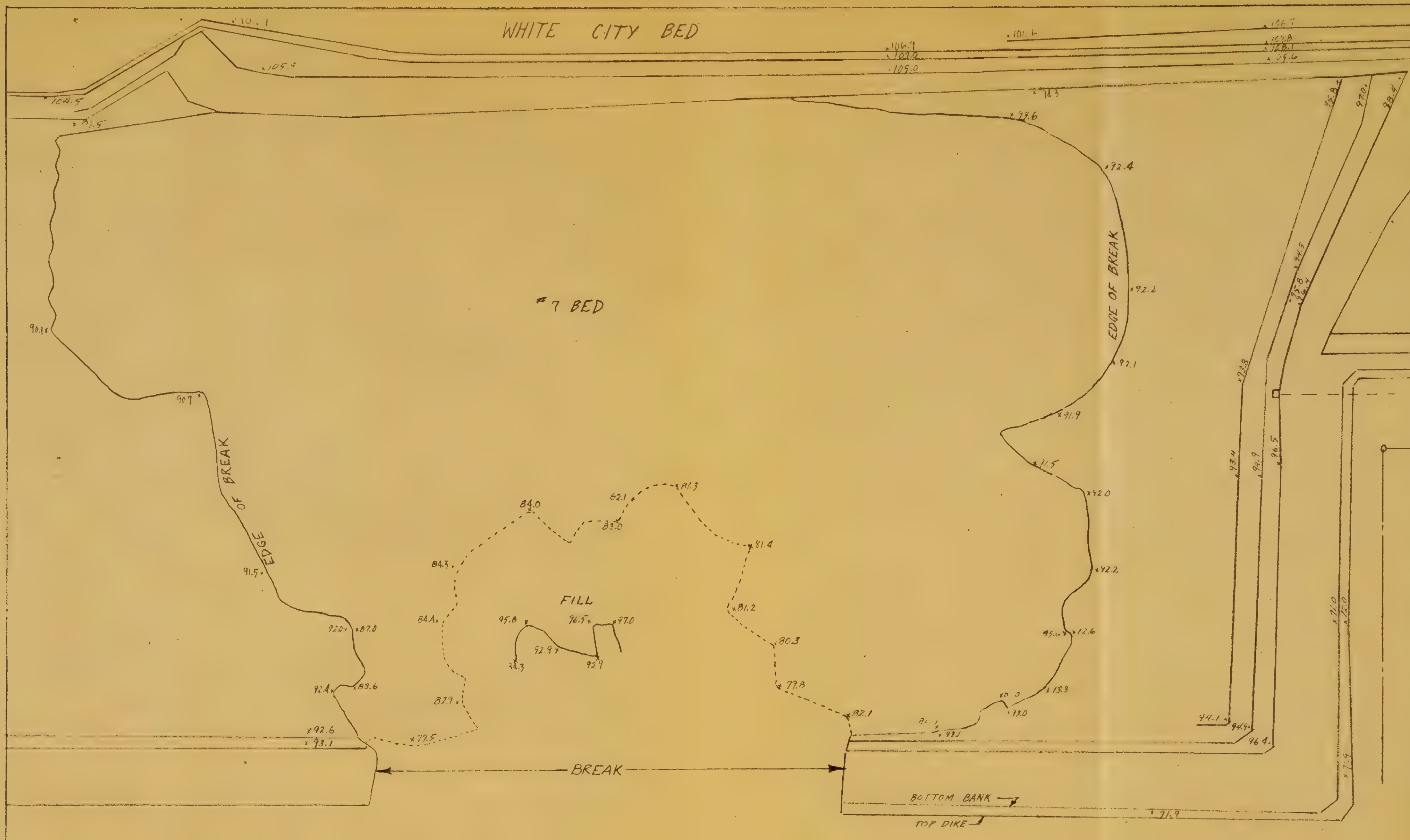
On October 1, 1955 the surcharge was completed to 1.5 feet above grade and let stand until the following spring. In the spring, it was decided to remove the surcharge and proceed with construction since only minor settlements were being observed at that time.

The design safety factor was 1.25. $C = 200$ PSF and $\varnothing = 25^{\circ}$ was assumed for the marl layer. $C = 200$ PSF and $\varnothing = 11^{\circ}$ was assumed for the silt and clay layer. The analysis also considered a full depth tension crack. Triaxial tests showed that the marl developed its full strength at a strain of 8-10% with a resulting drop of strength of approximately 20% at a strain of 5%. The stress-strain curves for the silt and clay were not run out beyond 5% to determine the loss of strength with greater strain.

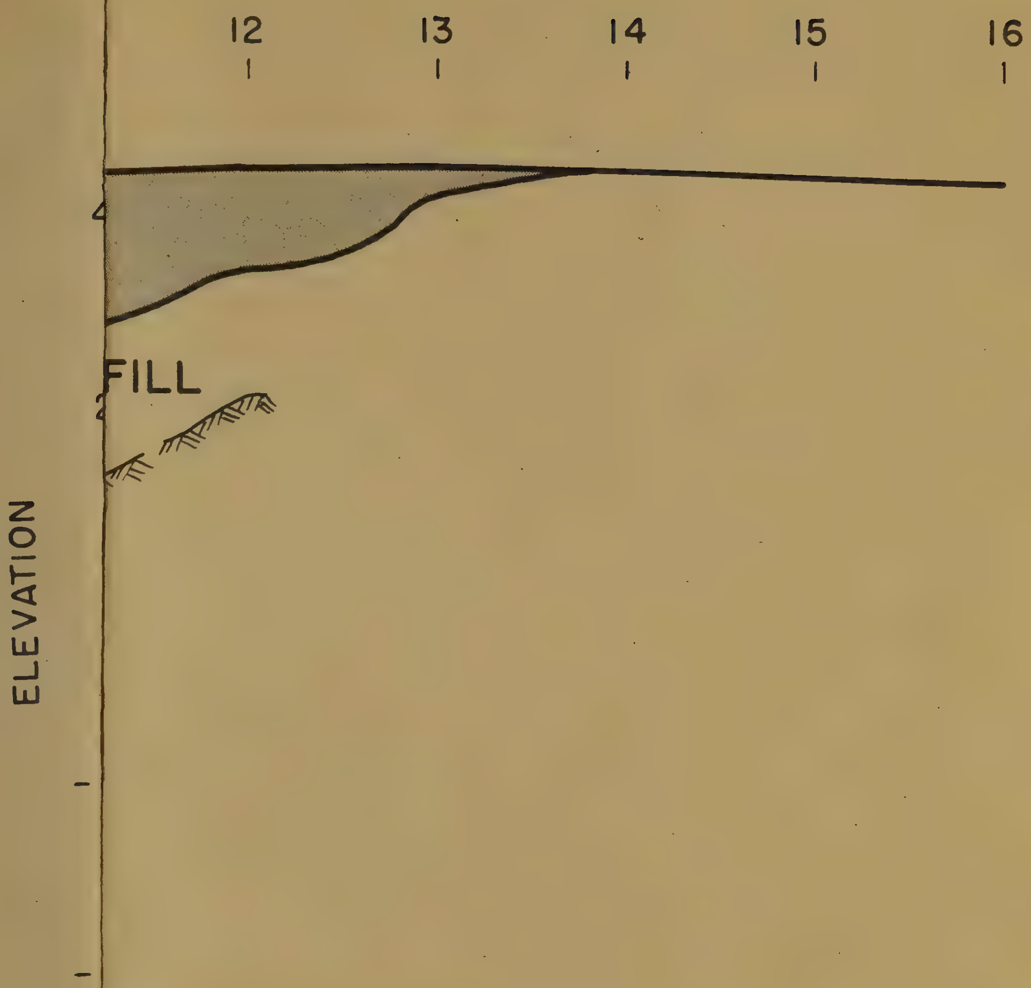
WM:HT



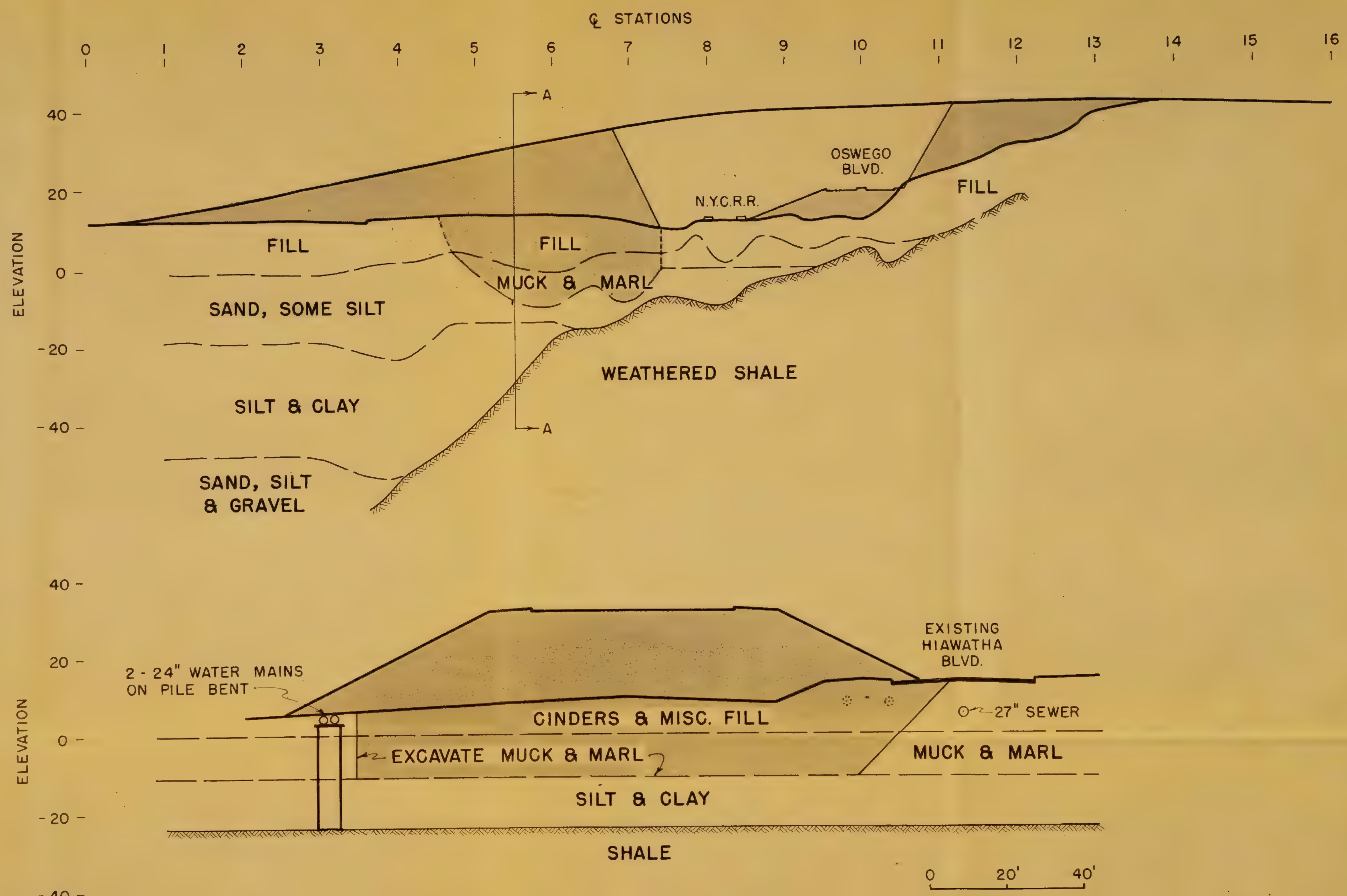
OLVAY SLUDGE BED FAILURE
YRACUSE STATE FAIR GROUNDS
ST. 3 ONONDAGA CO. 6/10/57



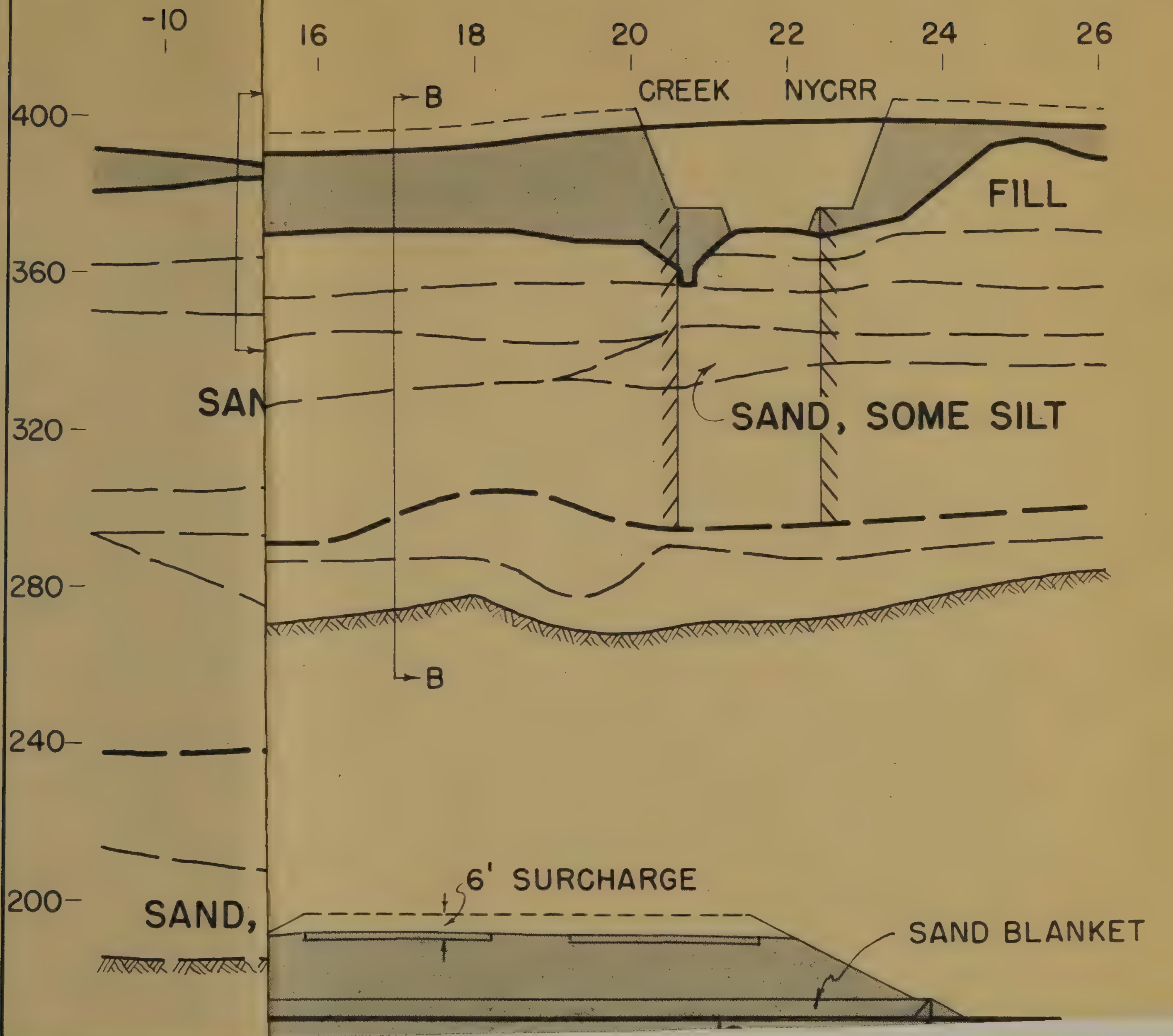
PLAN OF SOLVAY SLUDGE BED FAILURE
VICINITY OF SYRACUSE STATE FAIR GROUNDS
DIST. 3 ONONDAGA CO. 6/10/57


12
13
14
15
16
FILL
ELEVATION

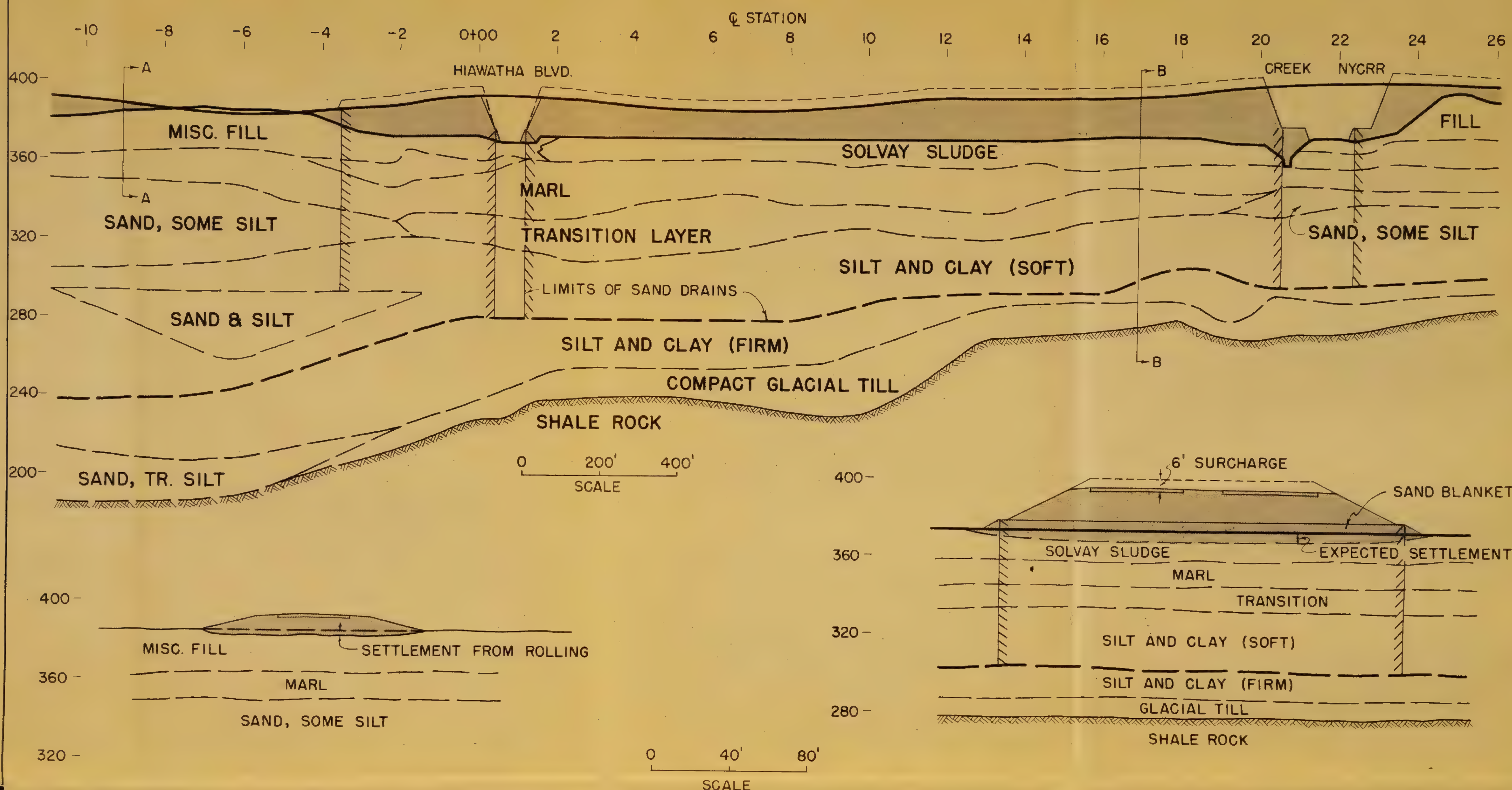
HIAWATHA BLVD. OVER OSWEGO BLVD.



SECTION A·A



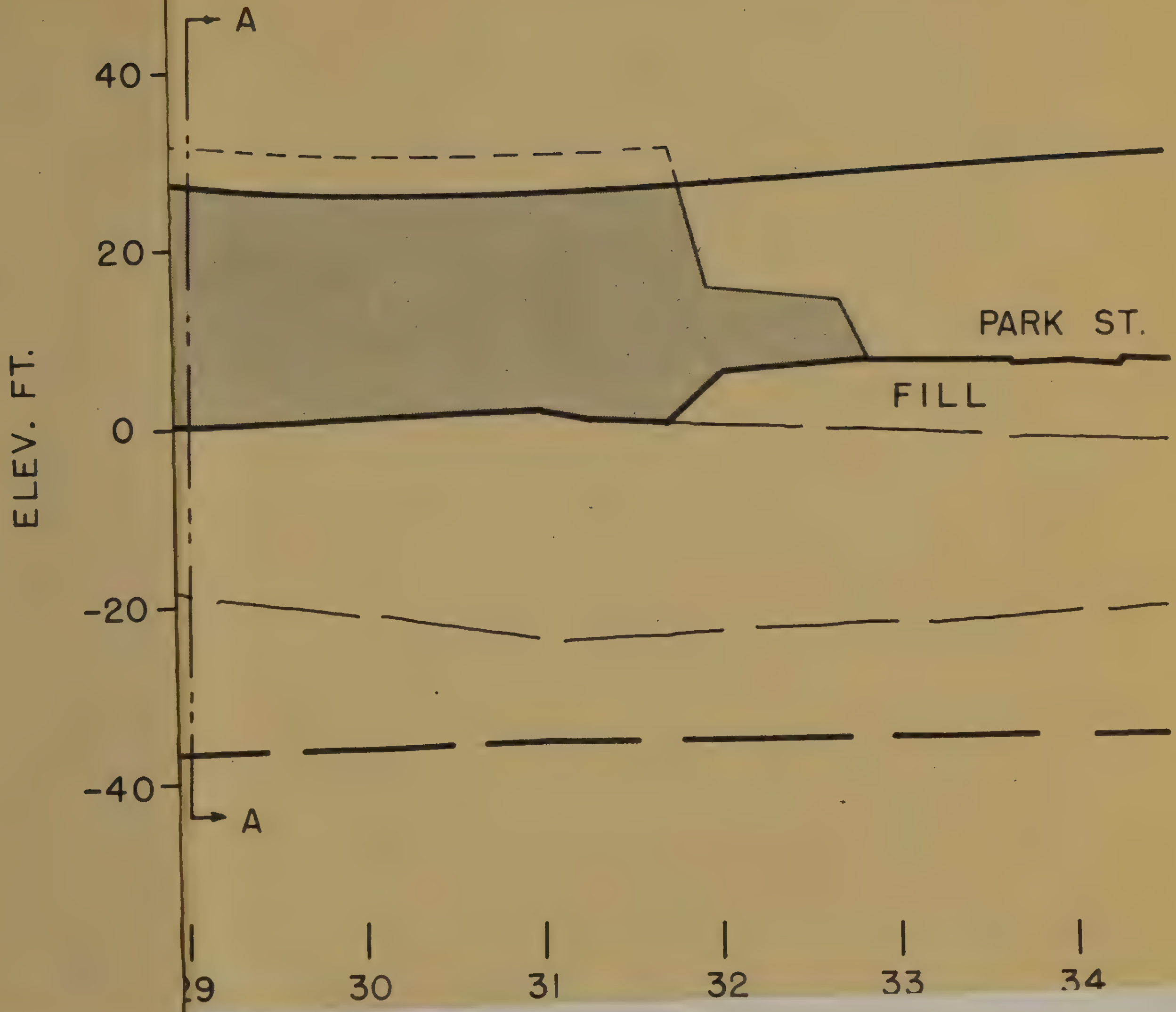
-10
16
18
20
22
24
26
400
360
320
280
240
200
B
B
CREEK
NYCRR
FILL
SAN
SAND, SOME SILT
SAND,
6' SURCHARGE
SAND BLANKET



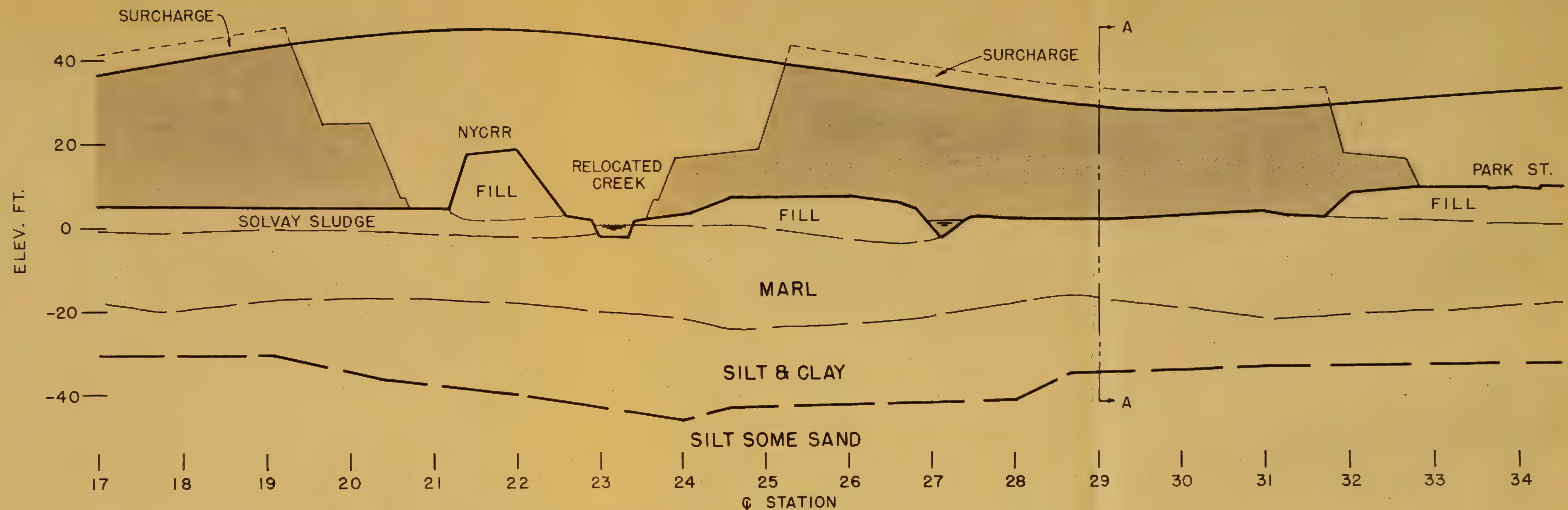
SECTION A-A - HEAVY ROLLING AREA

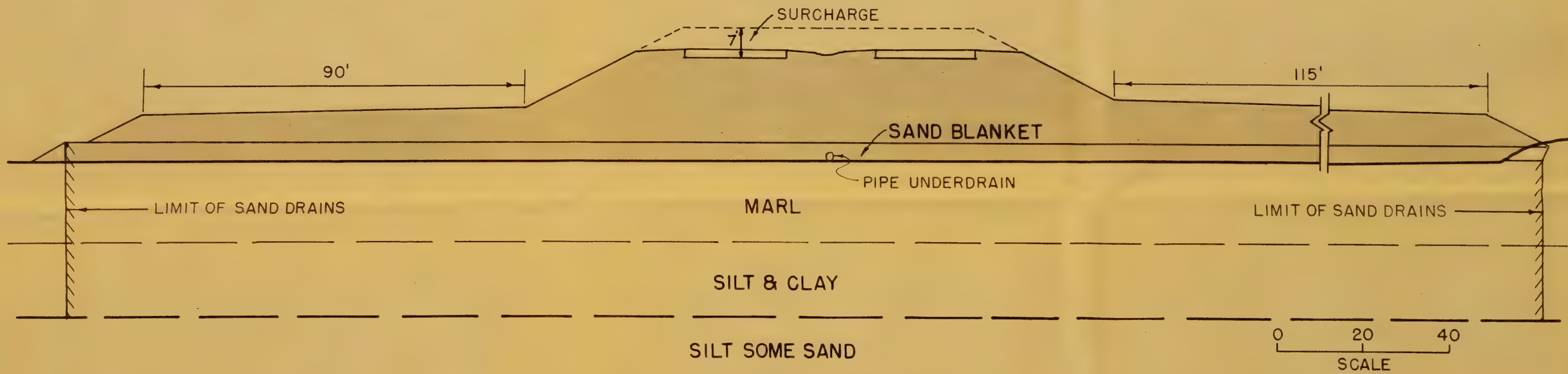
SECTION B-B - SAND DRAIN AREA

LAKE ONONDAGA WEST SHORE DEVELOPMENT
VICINITY OF HIAWATHA BLVD.

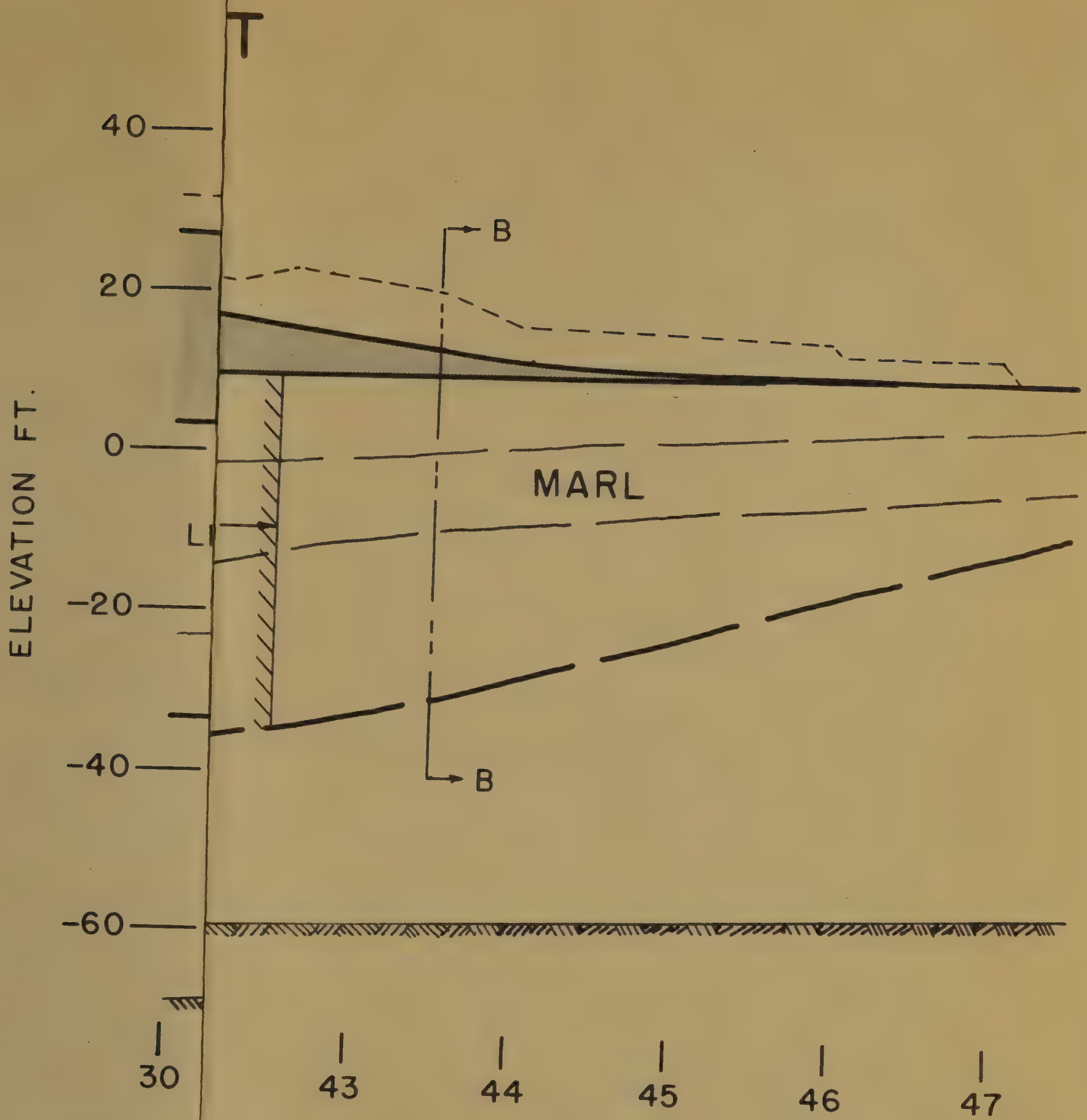

A
A
40
20
0
-20
-40
ELEV. FT.
PARK ST.
FILL
29
30
31
32
33
34

OSWEGO BOULEVARD
HIAWATHA BLVD. TO PARK ST.



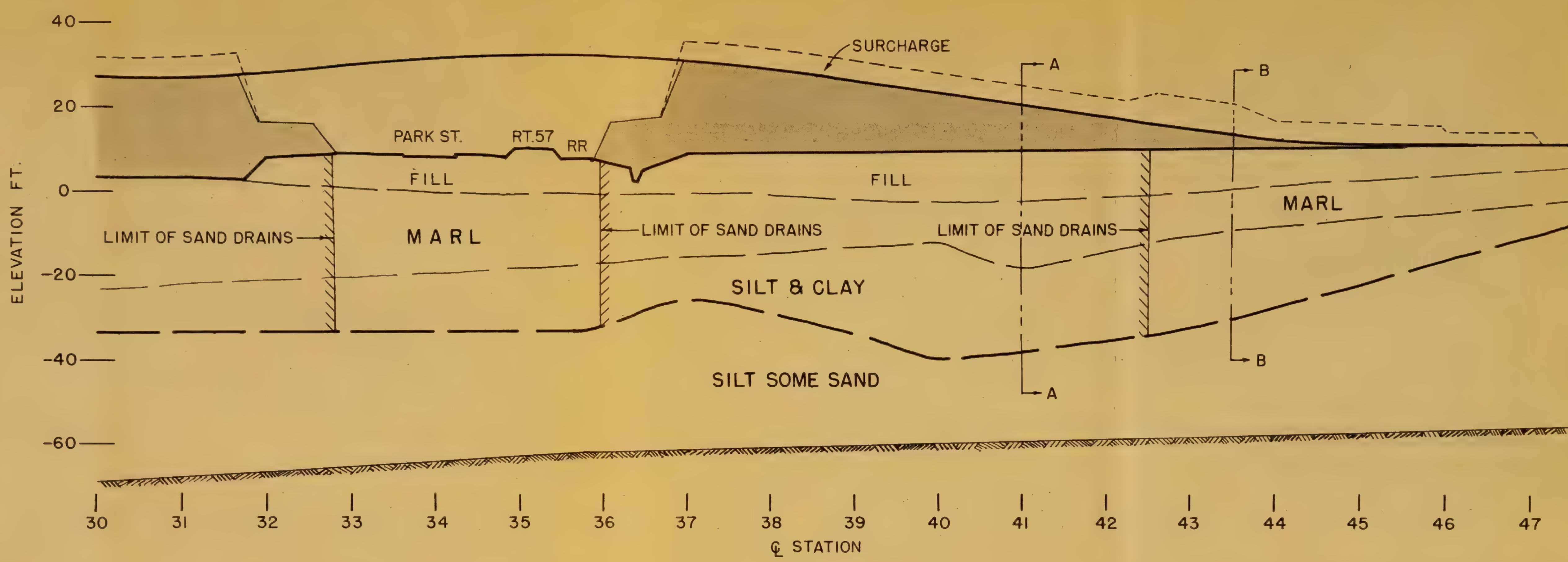


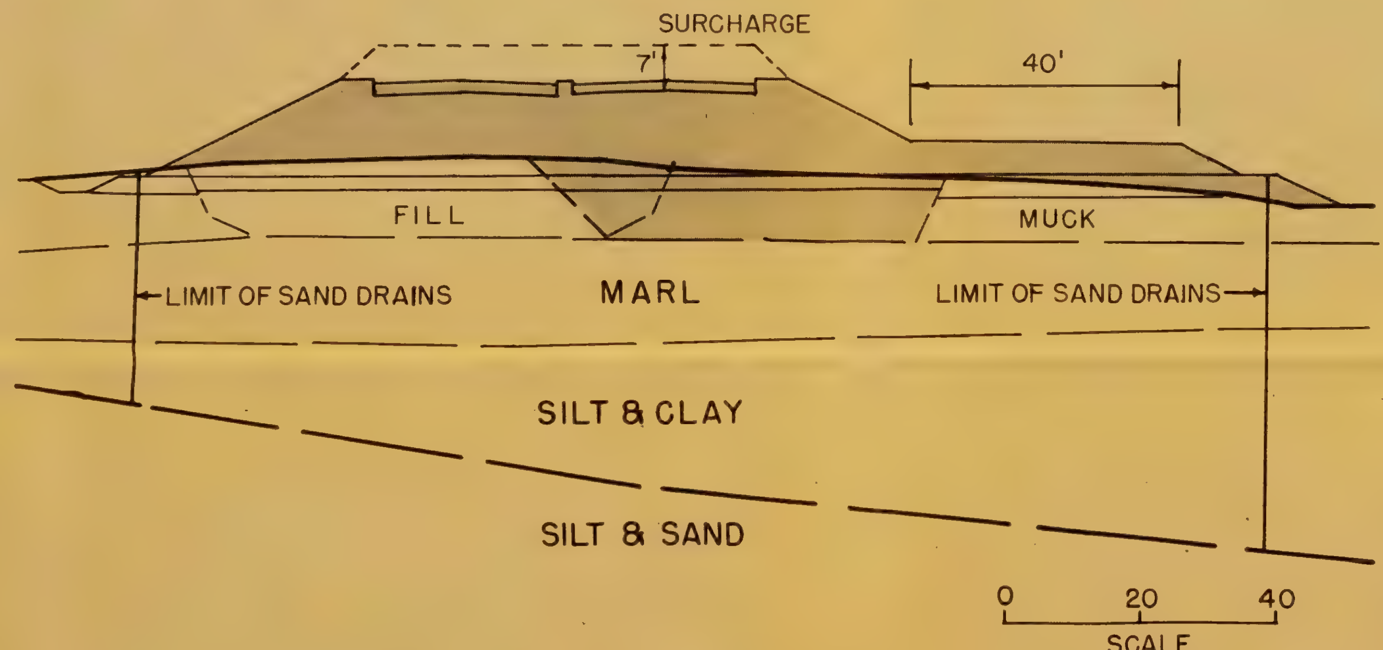
SECTION A-A



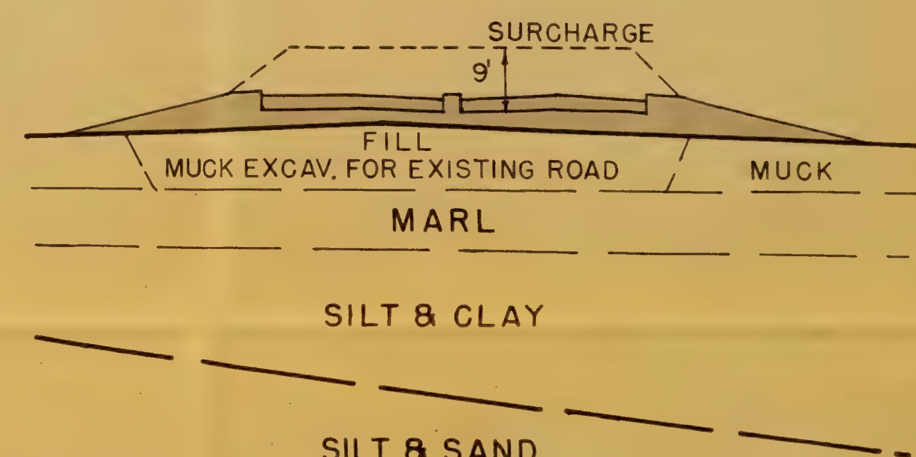
T
40
20
0
-20
-40
-60
ELEVATION FT.
B
B
MARL
L
30
43
44
45
46
47

OSWEGO BLVD. NORTH OF PARK STREET





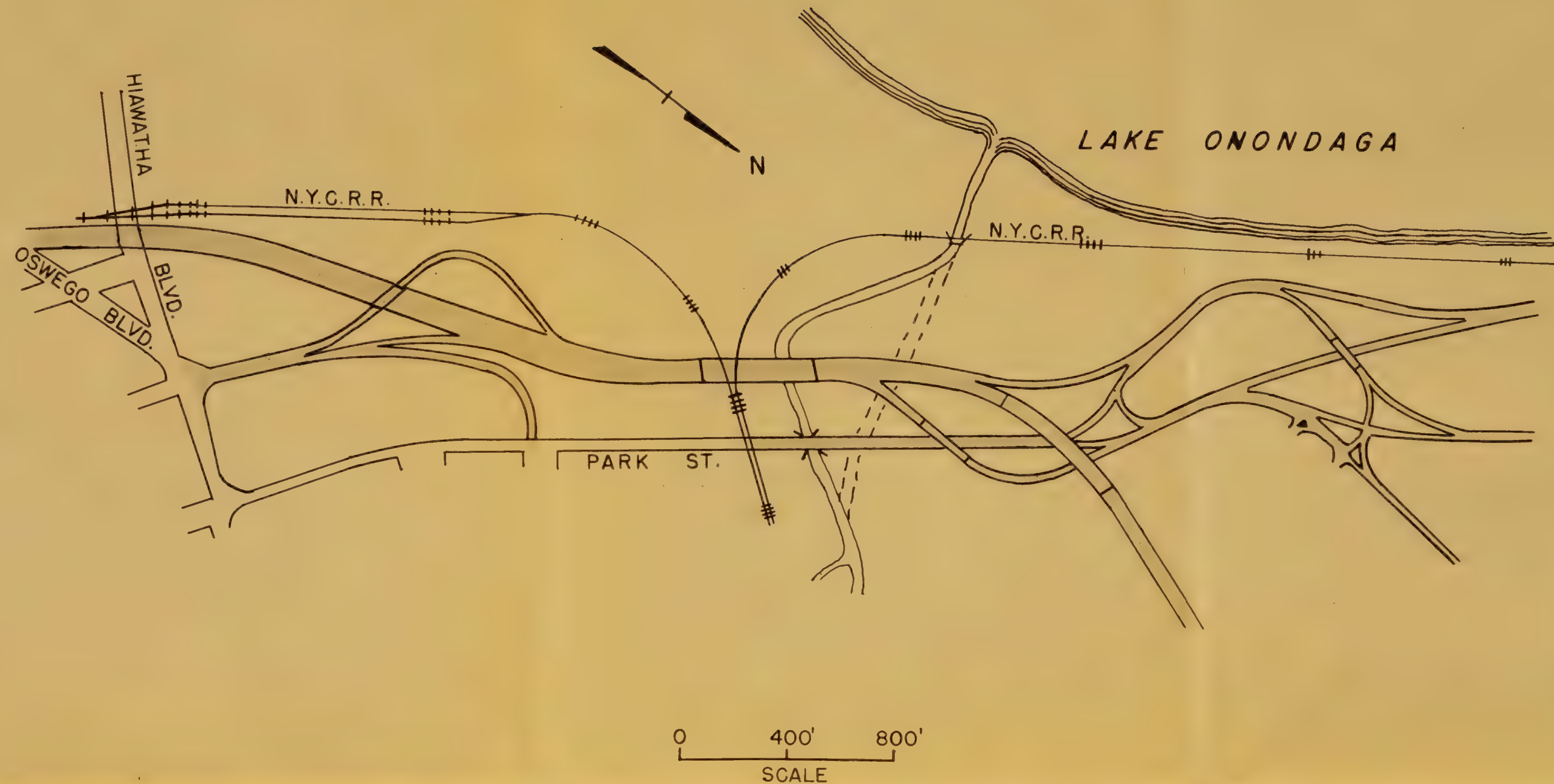
SECTION A-A

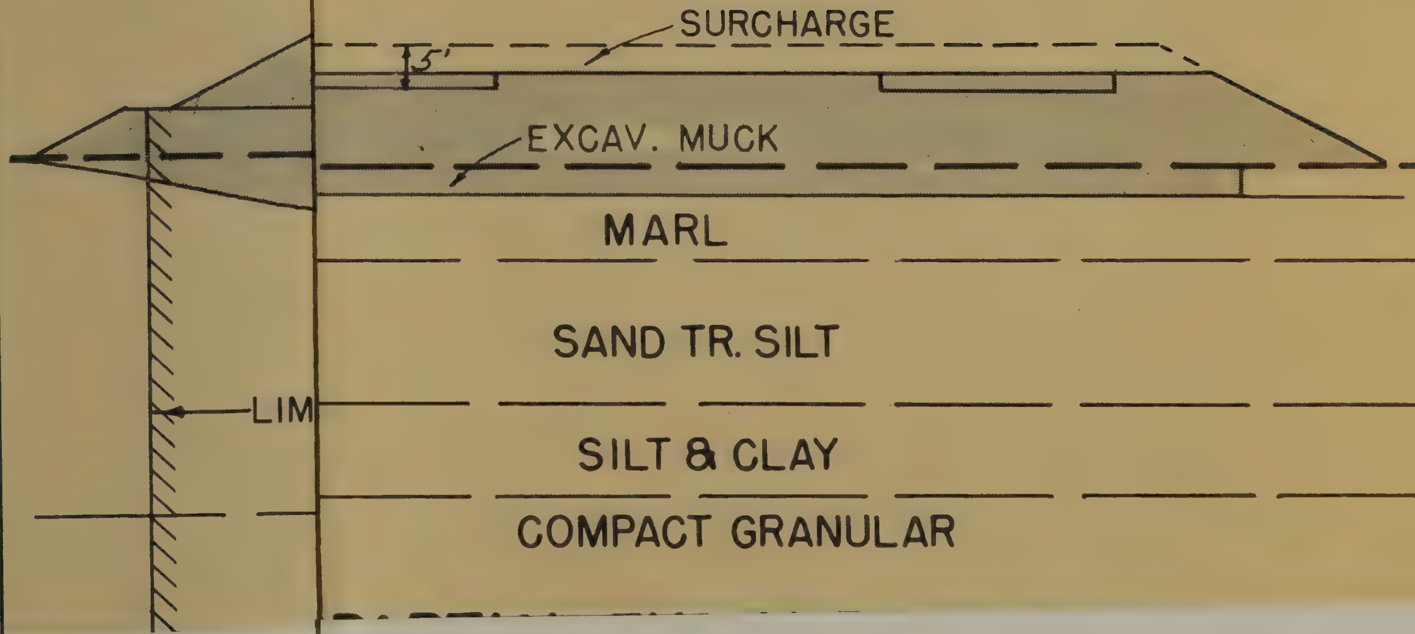


SECTION B-B

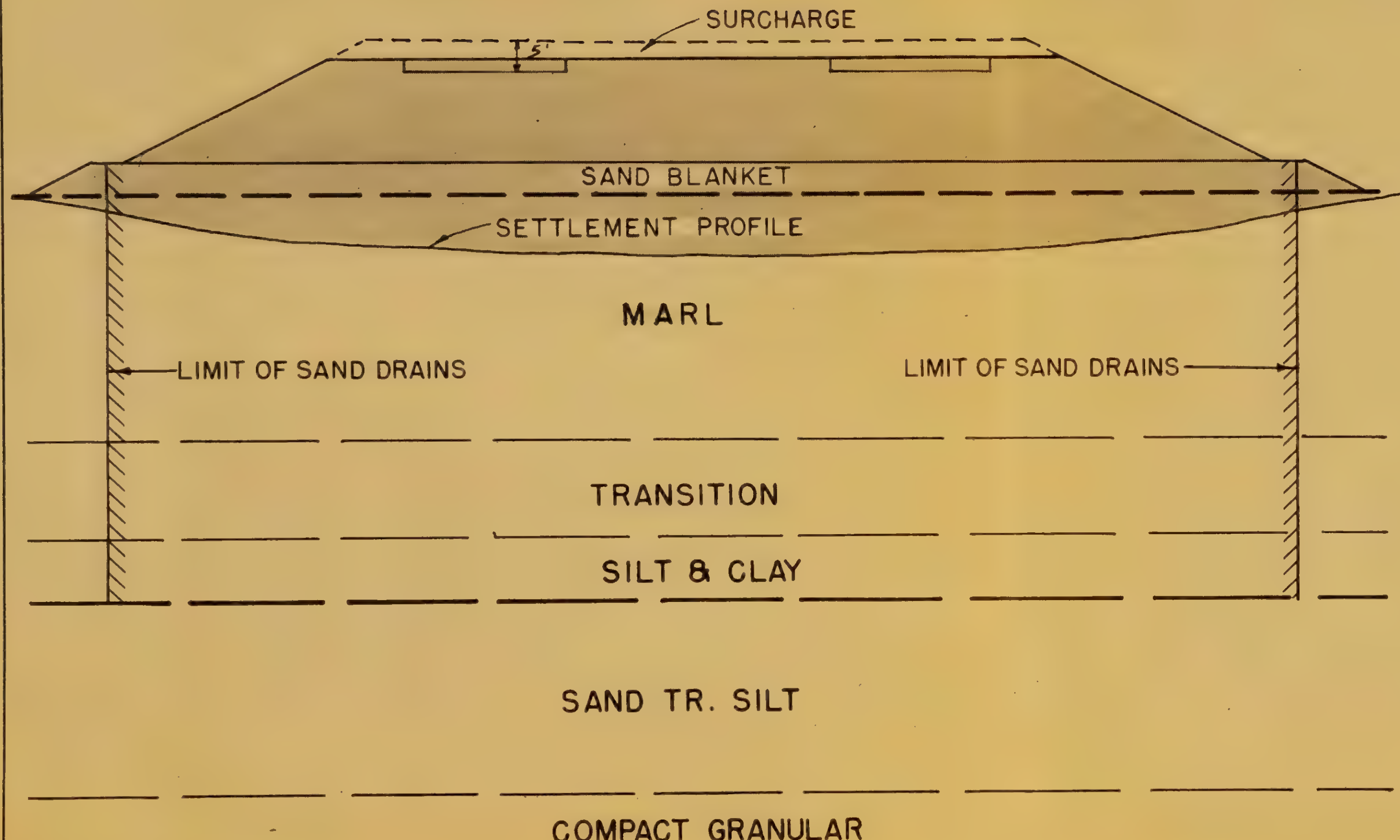
ONONDAGA

OSWEGO BLVD. LOCATION PLAN

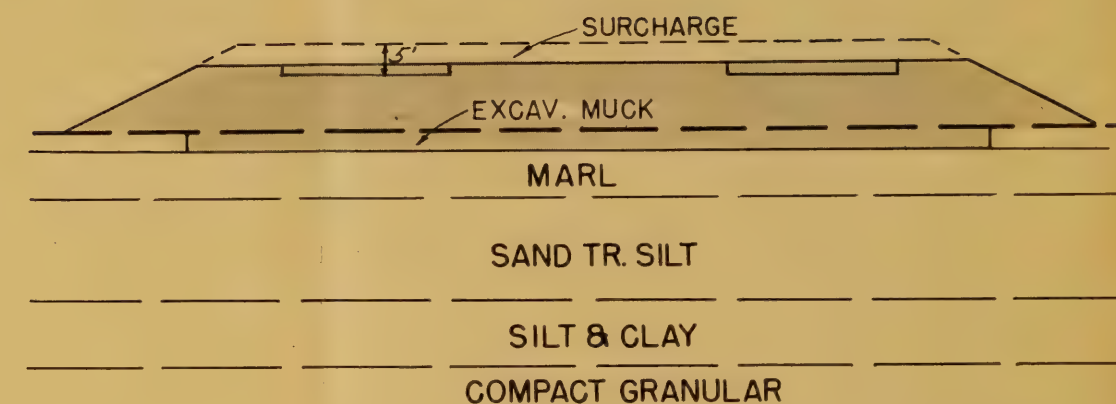



SURCHARGE
5'
EXCAV. MUCK
MARL
SAND TR. SILT
LIM
SILT & CLAY
COMPACT GRANULAR

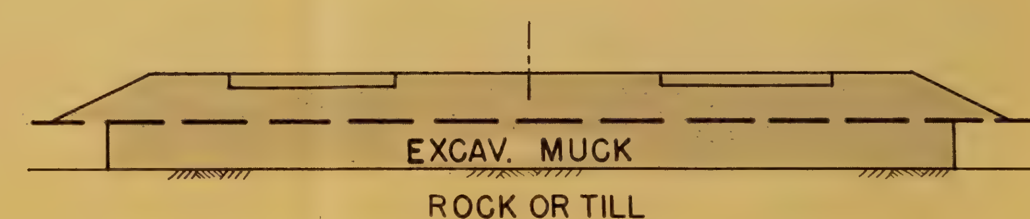
NEW YORK STATE THRUWAY
IN VICINITY OF
ONONDAGA LAKE OUTLET



SECTION A-A SAND DRAIN & SURCHARGE



SECTION B-B PARTIAL EXCAVATION & SURCHARGE



SECTION C-C COMPLETE EXCAVATION

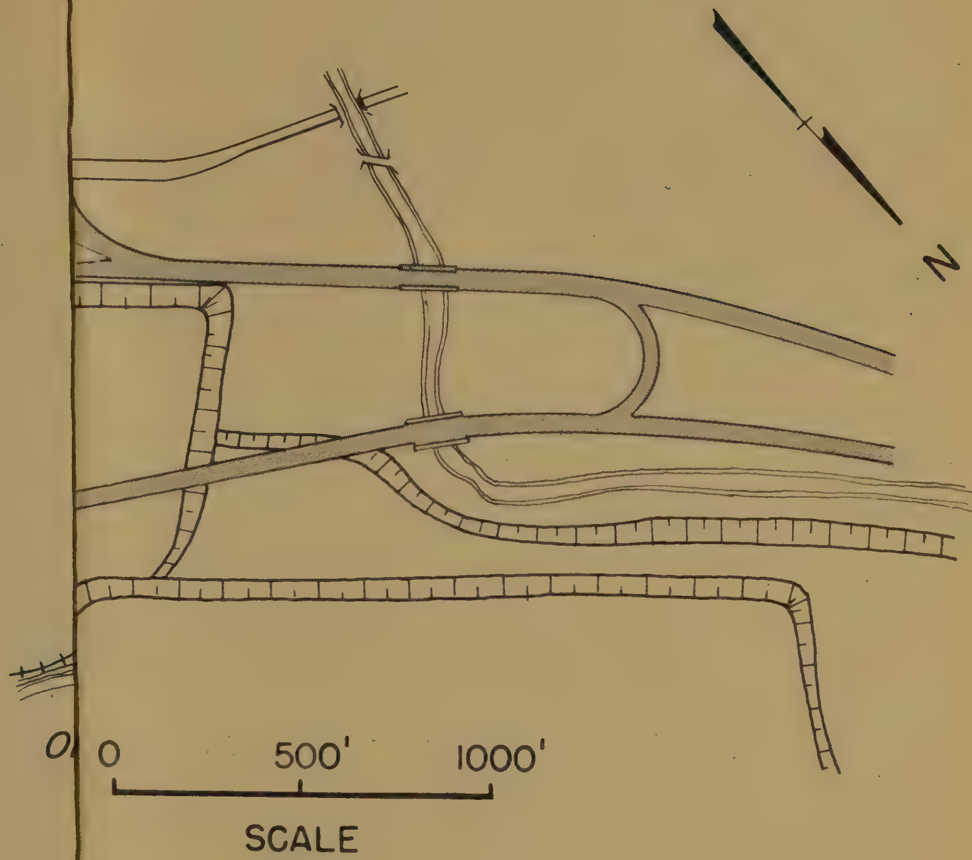
0 20' 40'

SCALE

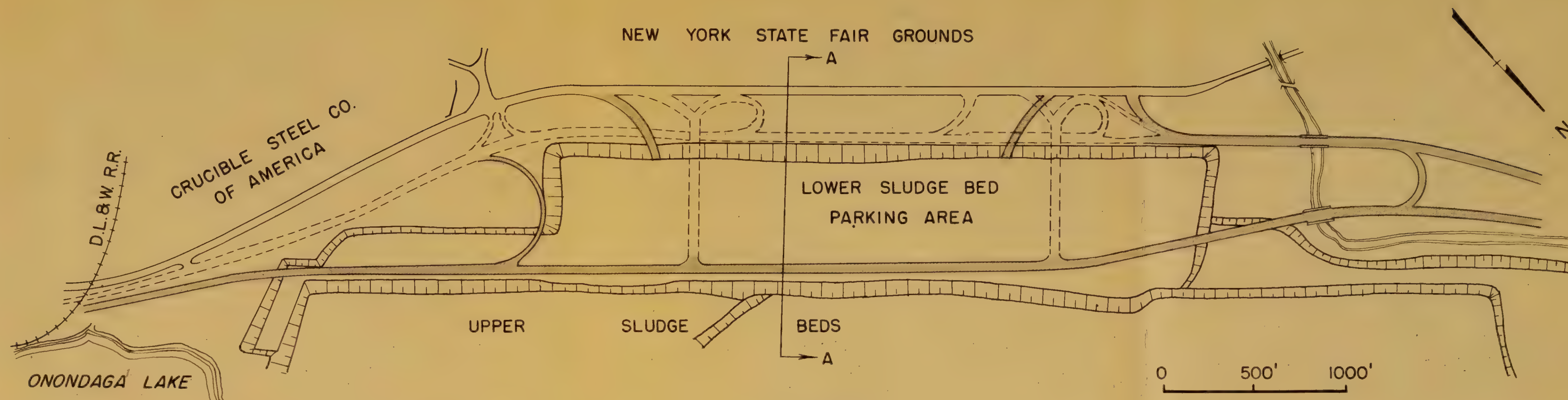
TYPICAL SECTIONS

MENT

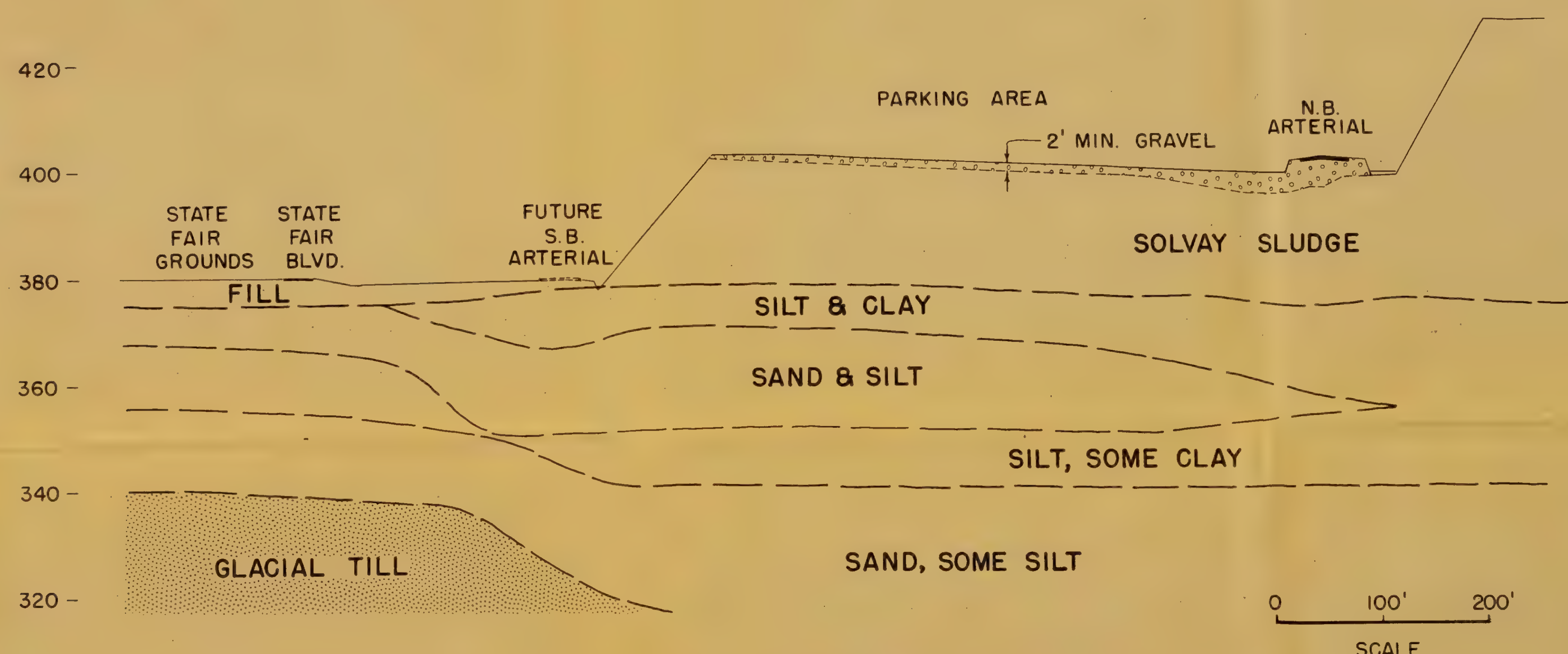
S



LAKE ONONDAGA WEST SHORE DEVELOPMENT
VICINITY OF STATE FAIR GROUNDS



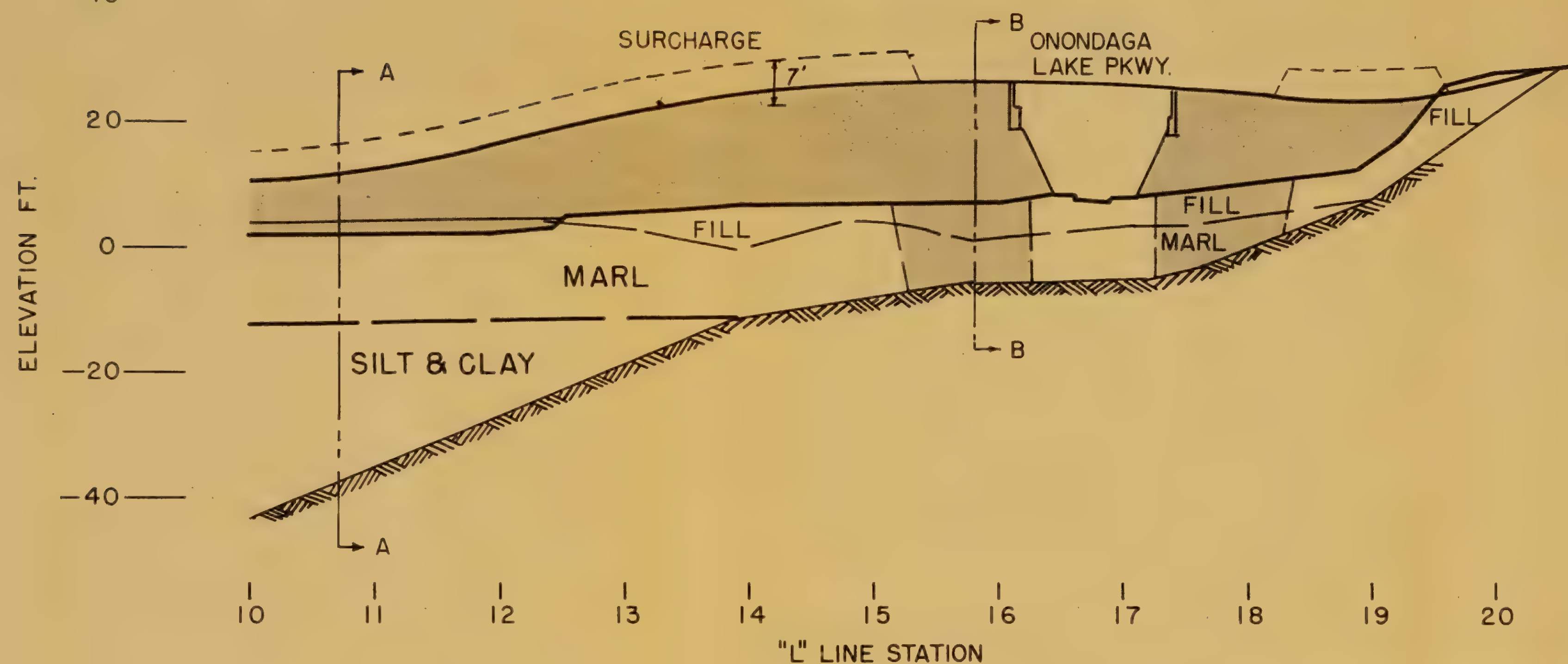
PLAN

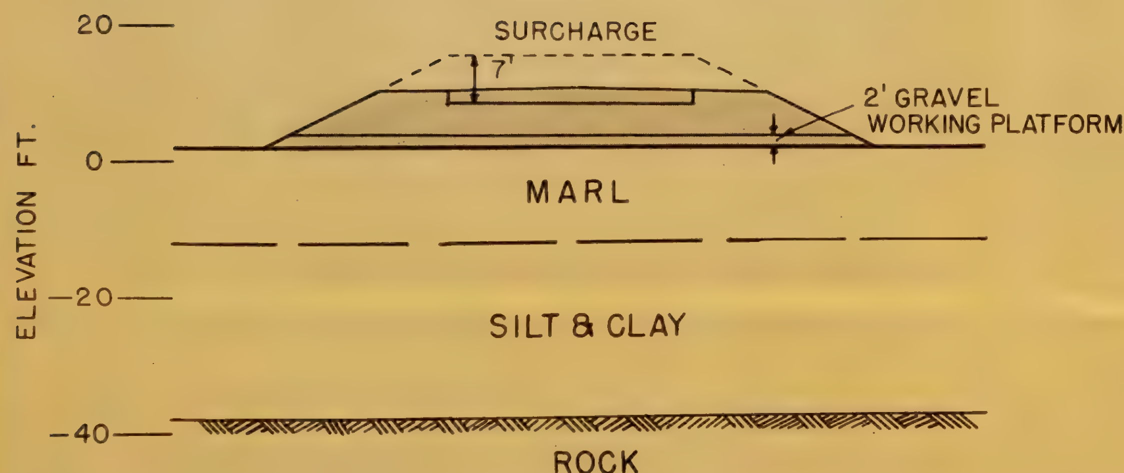


SECTION A·A

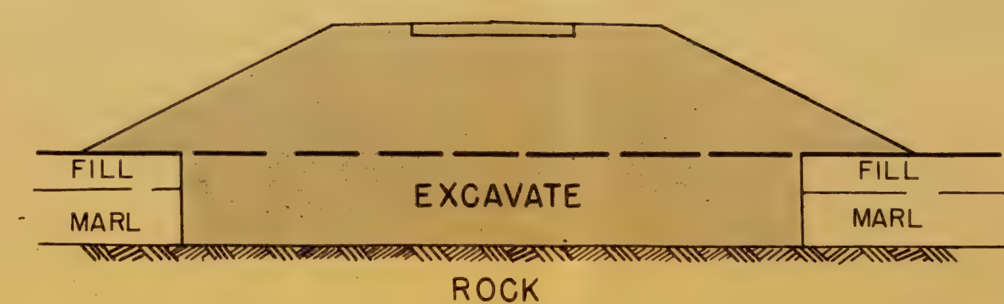
1
20

OSWEGO BOULEVARD
ACCESS RAMP NEAR PARK ST.

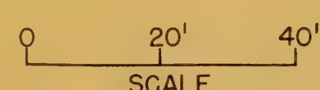


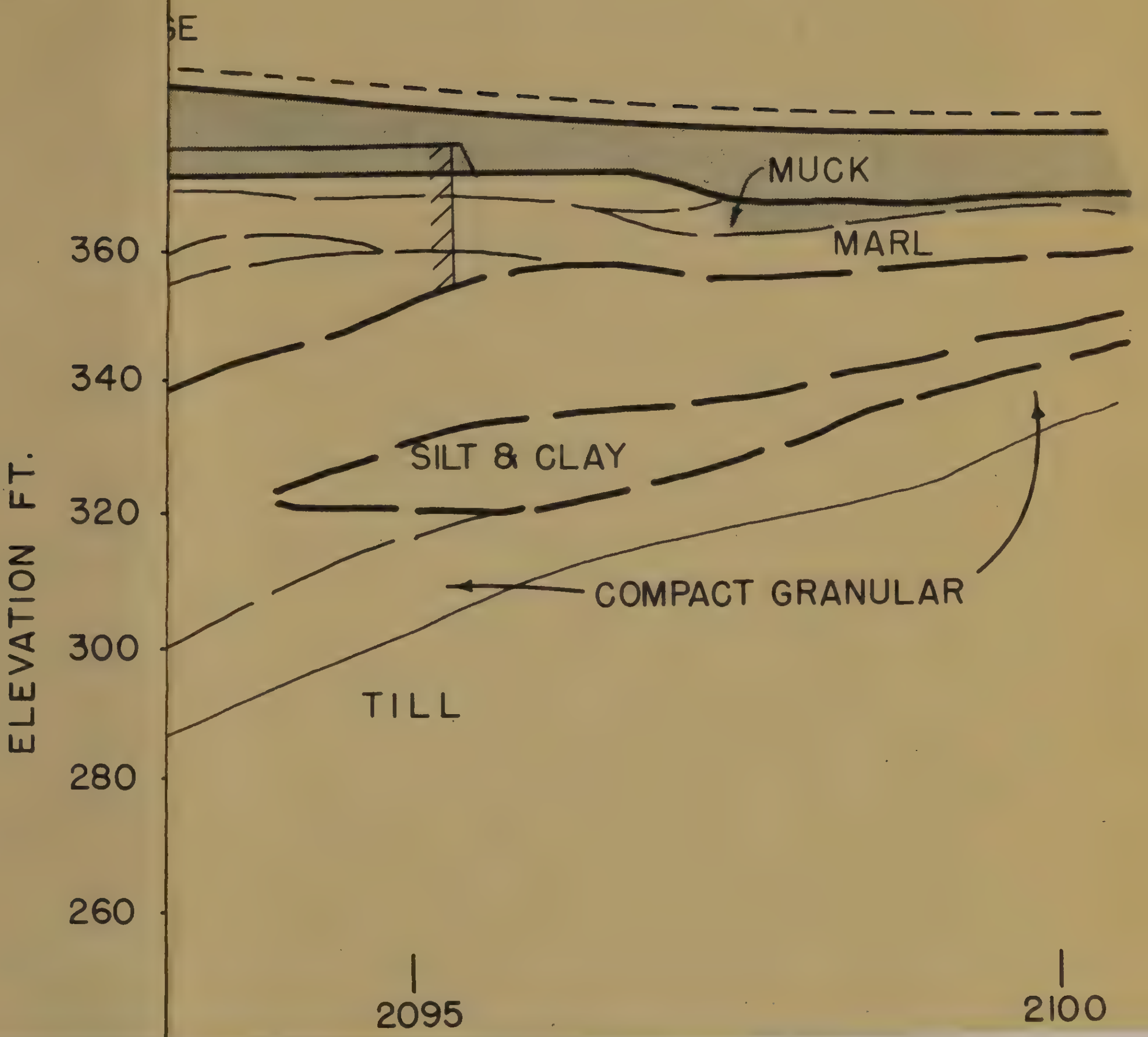


SECTION A-A



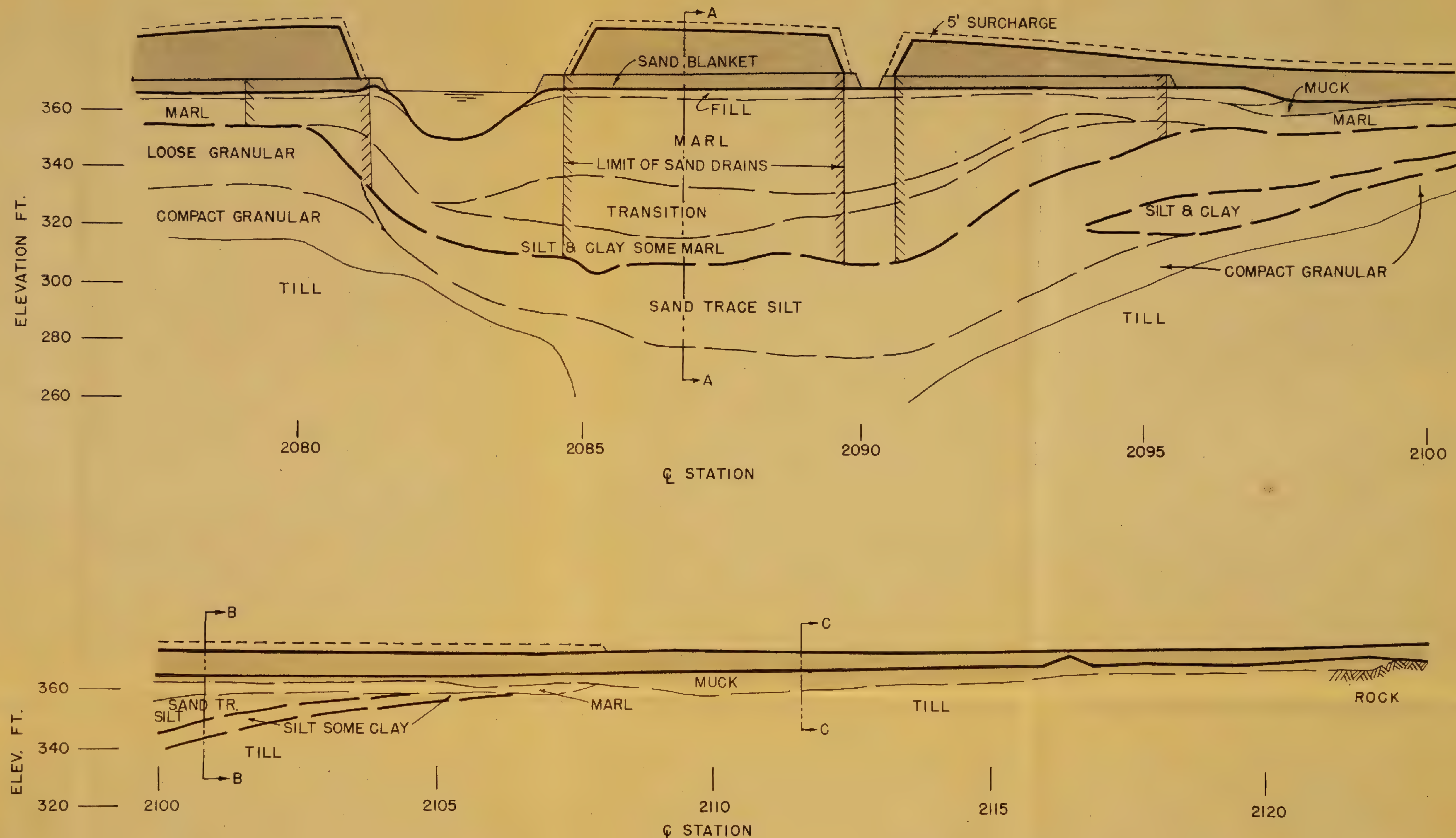
SECTION B-B





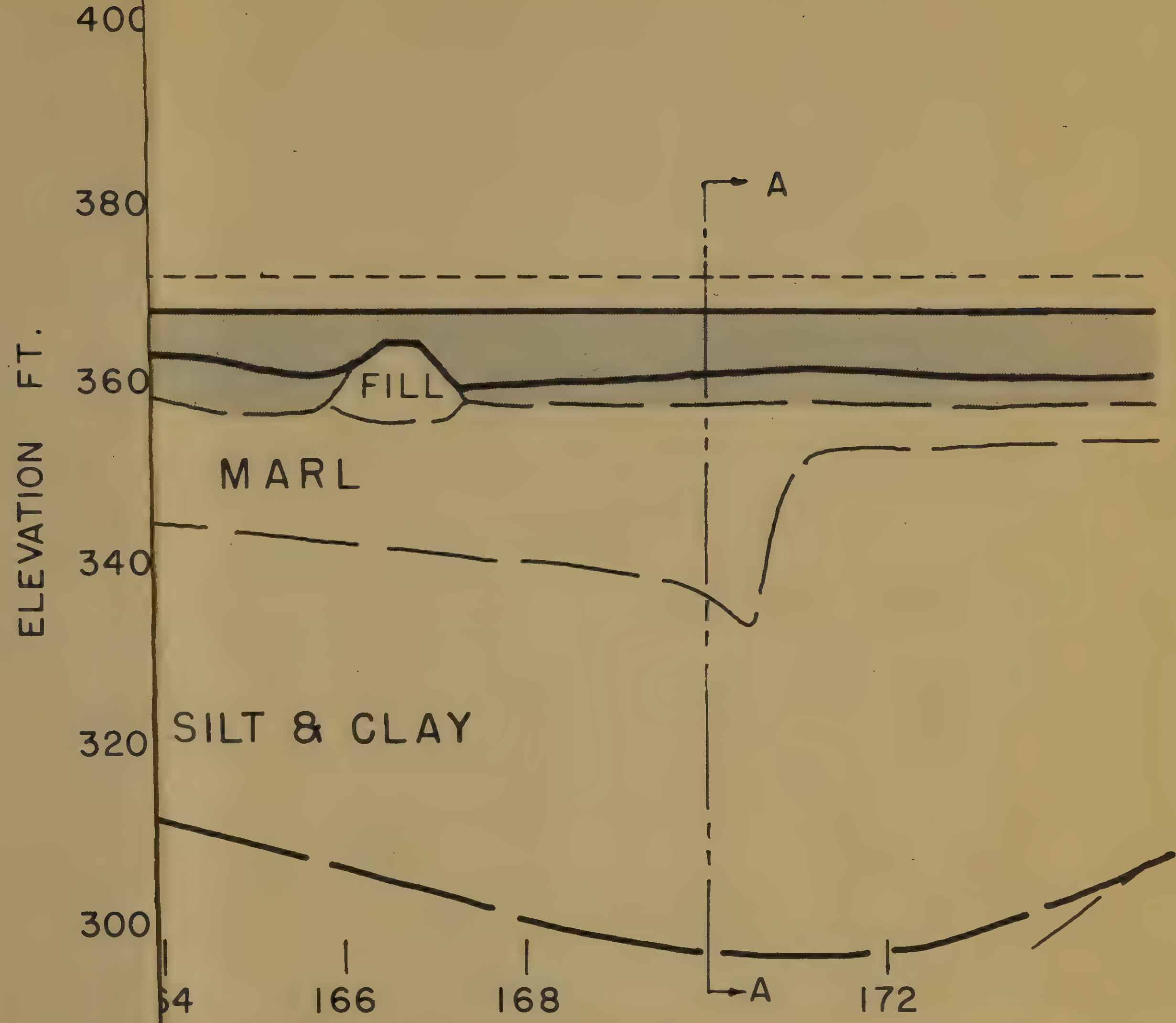
ELEVATION FT.
360
340
320
300
280
260
MUCK
MARL
SILT & CLAY
COMPACT GRANULAR
TILL
2095
2100

NEW YORK STATE THRUWAY
IN VICINITY OF ONONDAGA LAKE OUTLET



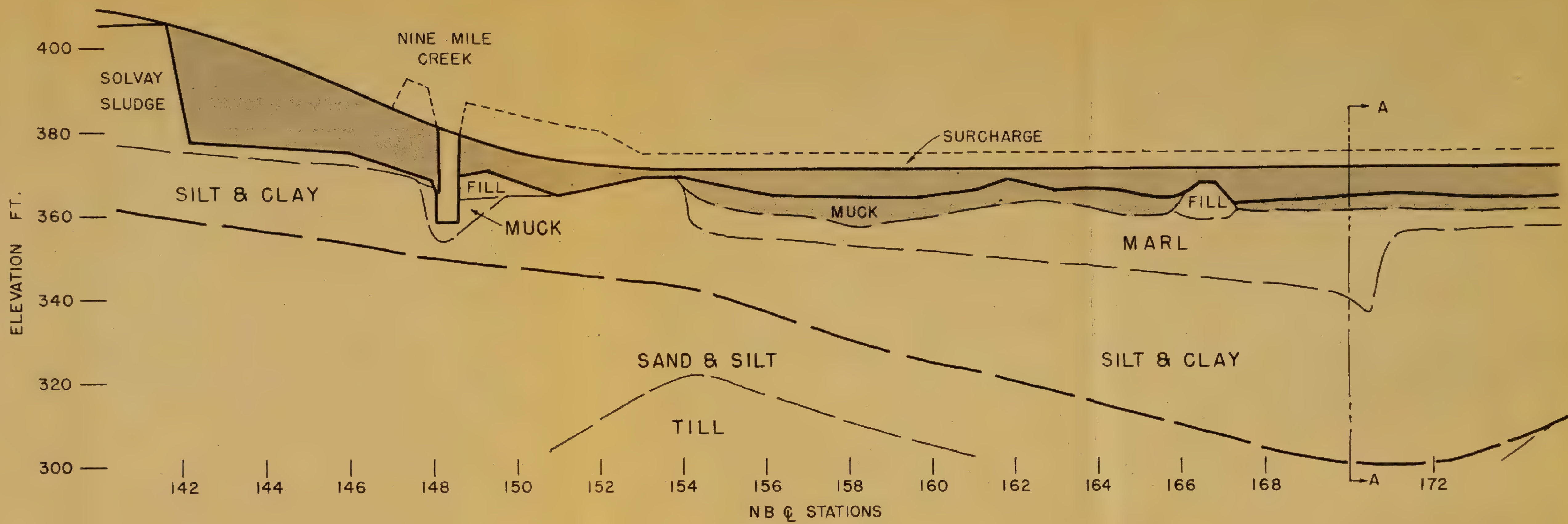
PROFILE

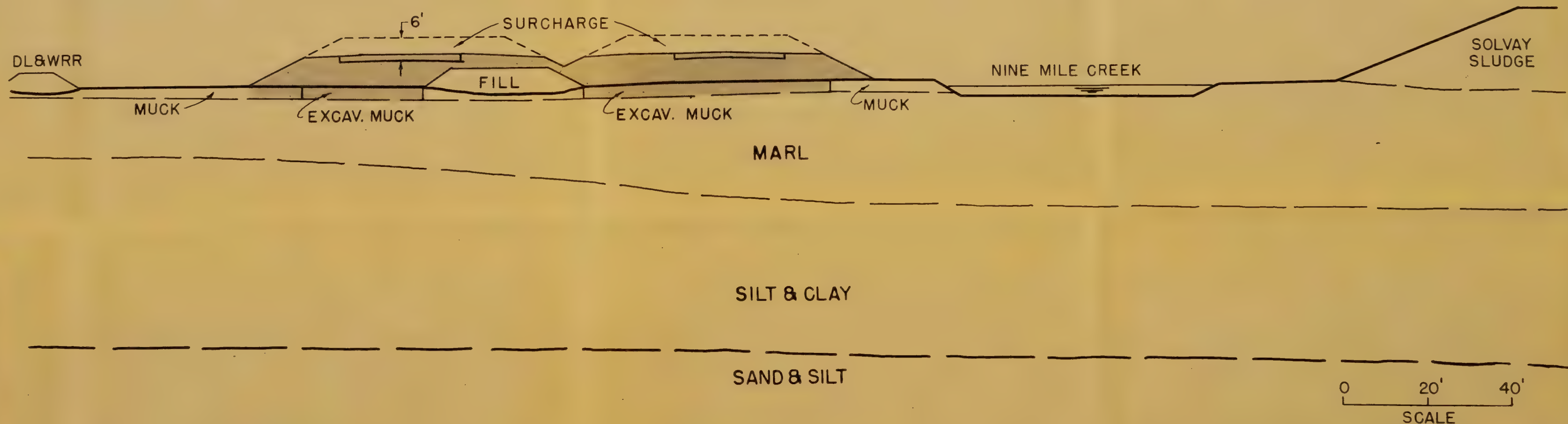
MENT



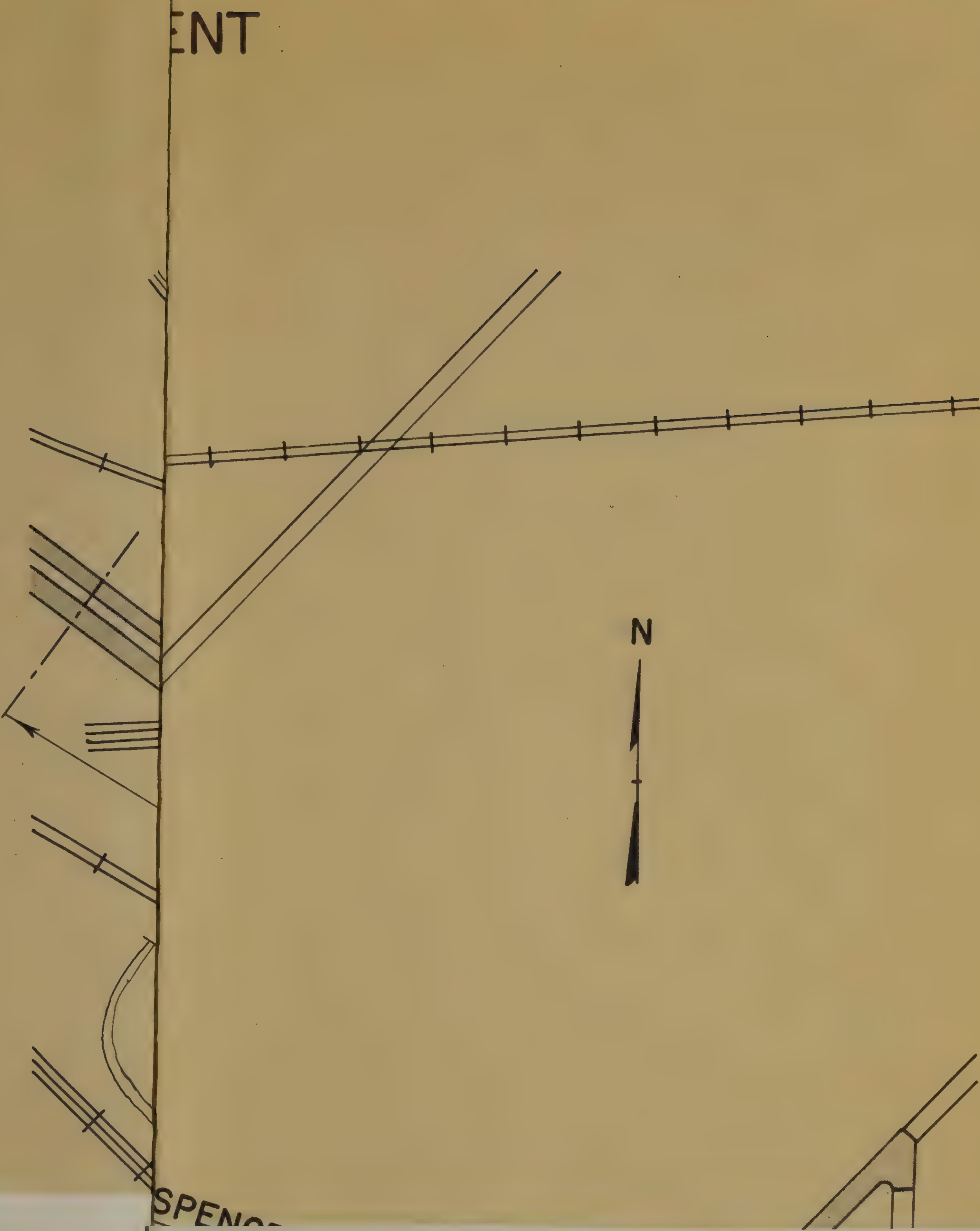
ELEVATION FT.
400
380
360
340
320
300
A
FILL
MARL
SILT & CLAY
A
166
168
172

LAKE ONONDAGA WEST SHORE DEVELOPMENT
NORTH OF STATE FAIR GROUNDS

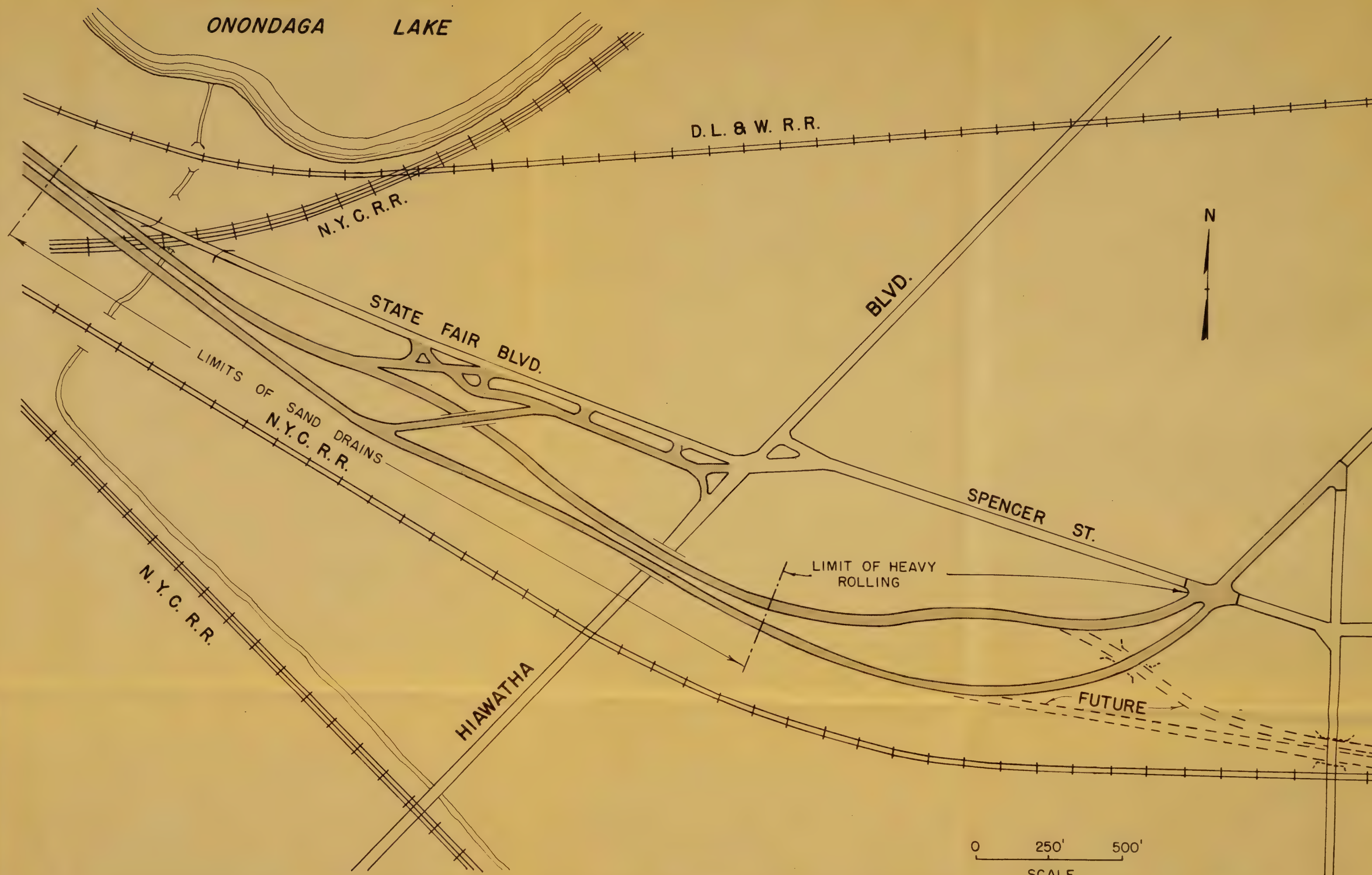




SECTION A-A

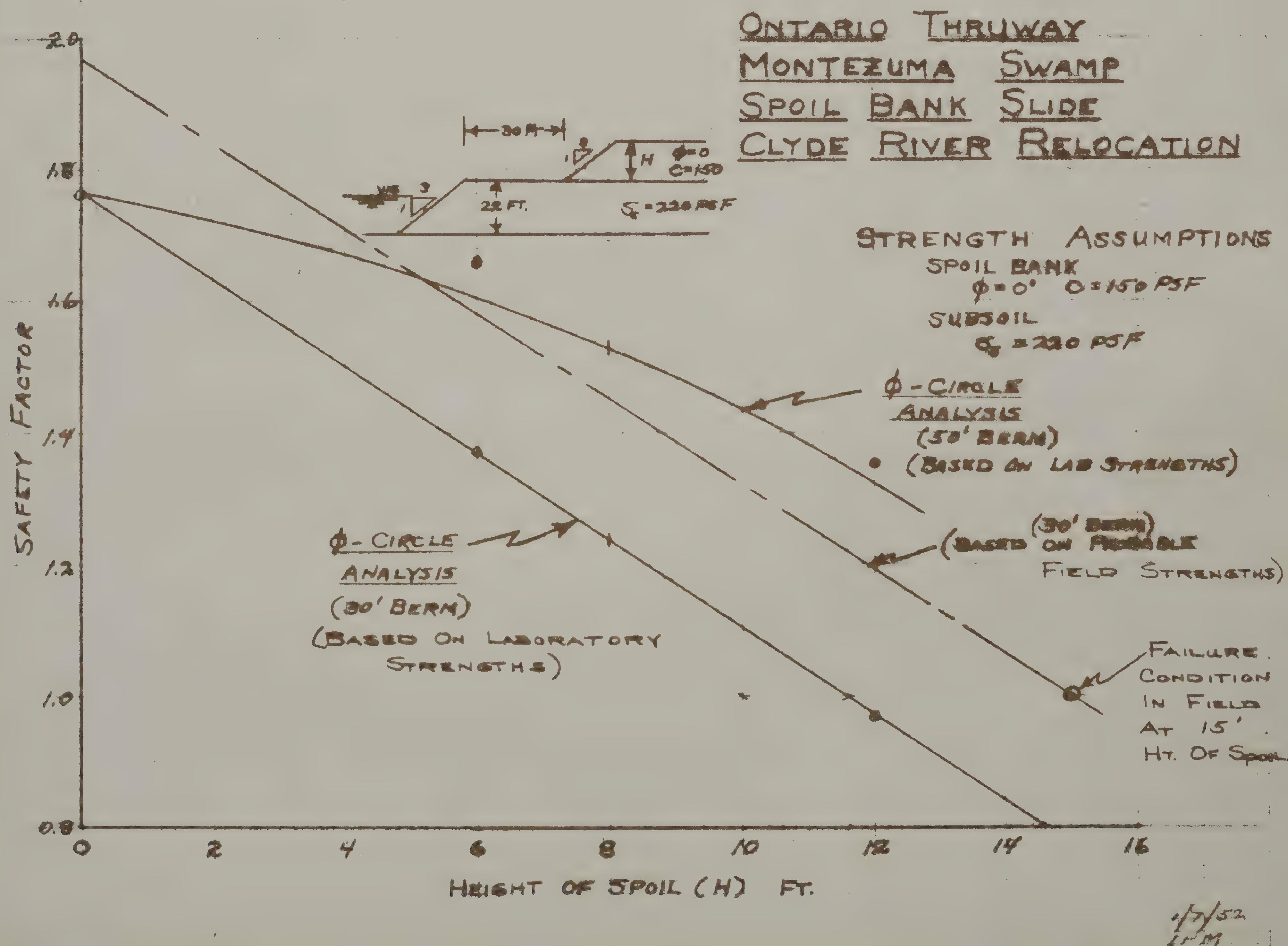

ENT
N

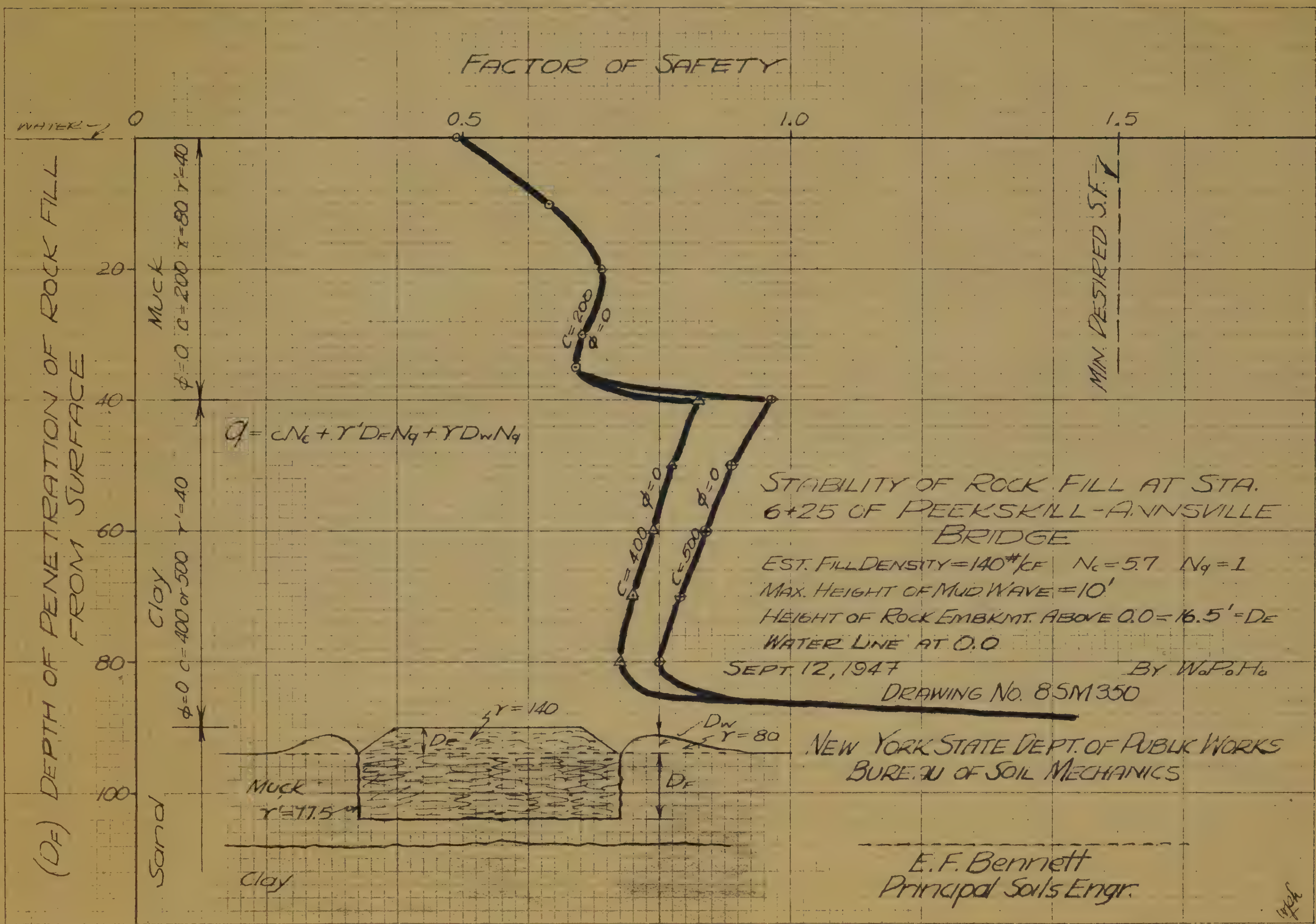
LAKE ONONDAGA WEST SHORE DEVELOPMENT
VICINITY OF HIAWATHA BLVD.



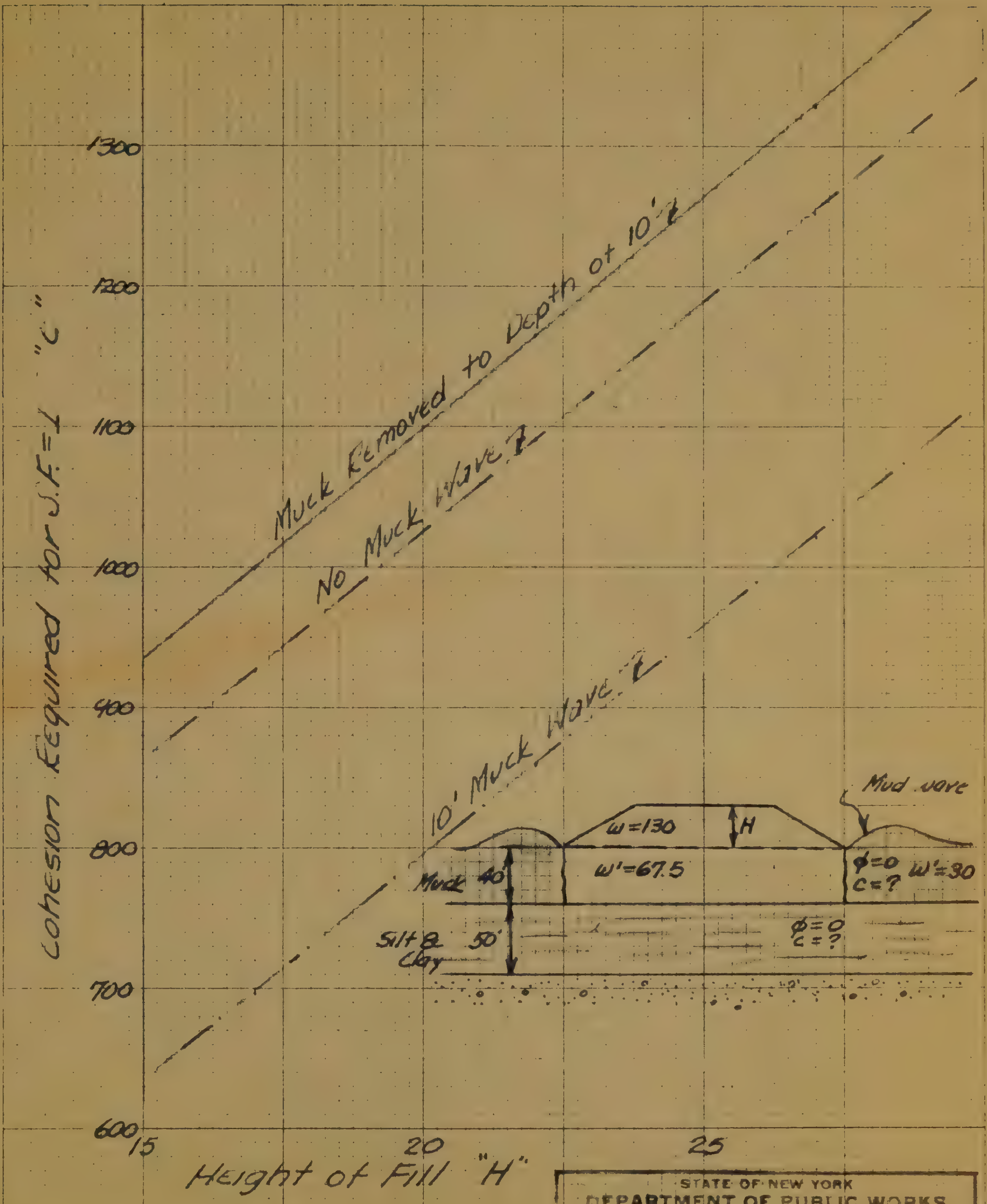
EA

LOCATION PLAN - SYRACUSE AREA


ONTARIO THRUWAY
MONTEZUMA SWAMP
SPOIL BANK SLIDE
CLYDE RIVER RELOCATION
STRENGTH ASSUMPTIONS
SPOIL BANK
φ = 0° C = 150 PSF
SUBSOIL
= 220 PSF
30 FT.
22 FT.
H
φ = 0
C = 150
= 220 PSF
φ - CIRCLE
ANALYSIS
(50' BERM)
(BASED ON LAB STRENGTHS)
(30' BERM)
(BASED ON PROBABLE
FIELD STRENGTHS)
φ - CIRCLE
ANALYSIS
(30' BERM)
(BASED ON LABORATORY
STRENGTHS)
FAILURE
CONDITION
IN FIELD
AT 15'
HT. OF SPOIL
SAFETY FACTOR
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0
2
4
6
8
10
12
14
16
HEIGHT OF SPOIL (H) FT.
1/7/52



KEUFFEL & ESSER CO. N. Y. NO. 359T-5G
FACTOR OF SAFETY
0
0.5
1.0
1.5
WATER
MIN. DESIRED S.F.
20
40
60
80
100
(DF) DEPTH OF PENETRATION OF ROCK FILL FROM SURFACE
Muck
ϕ=0 c=200 γ=80 γ'=40
Clay
ϕ=0 c=400 or 500 γ'=40
Sand
c=200 ϕ=0
c=400 ϕ=0
c=500 ϕ=0
$q = cN_c + \gamma' D_F N_q + \gamma D_W N_q$
STABILITY OF ROCK FILL AT STA. 6+25 OF PEEKSKILL-ANNSVILLE BRIDGE
EST. FILL DENSITY = 140#/CF   Nc = 5.7   Nq = 1
MAX. HEIGHT OF MUD WAVE = 10'
HEIGHT OF ROCK EMBKMT. ABOVE 0.0 = 16.5' = DE
WATER LINE AT 0.0
SEPT. 12, 1947
BY W.P.H.
DRAWING No. 8SM350
γ=140
DE
DW
γ=80
DF
Muck
γ'=77.5
Clay
NEW YORK STATE DEPT. OF PUBLIC WORKS
BUREAU OF SOIL MECHANICS
E. F. Bennett
Principal Soils Engr.



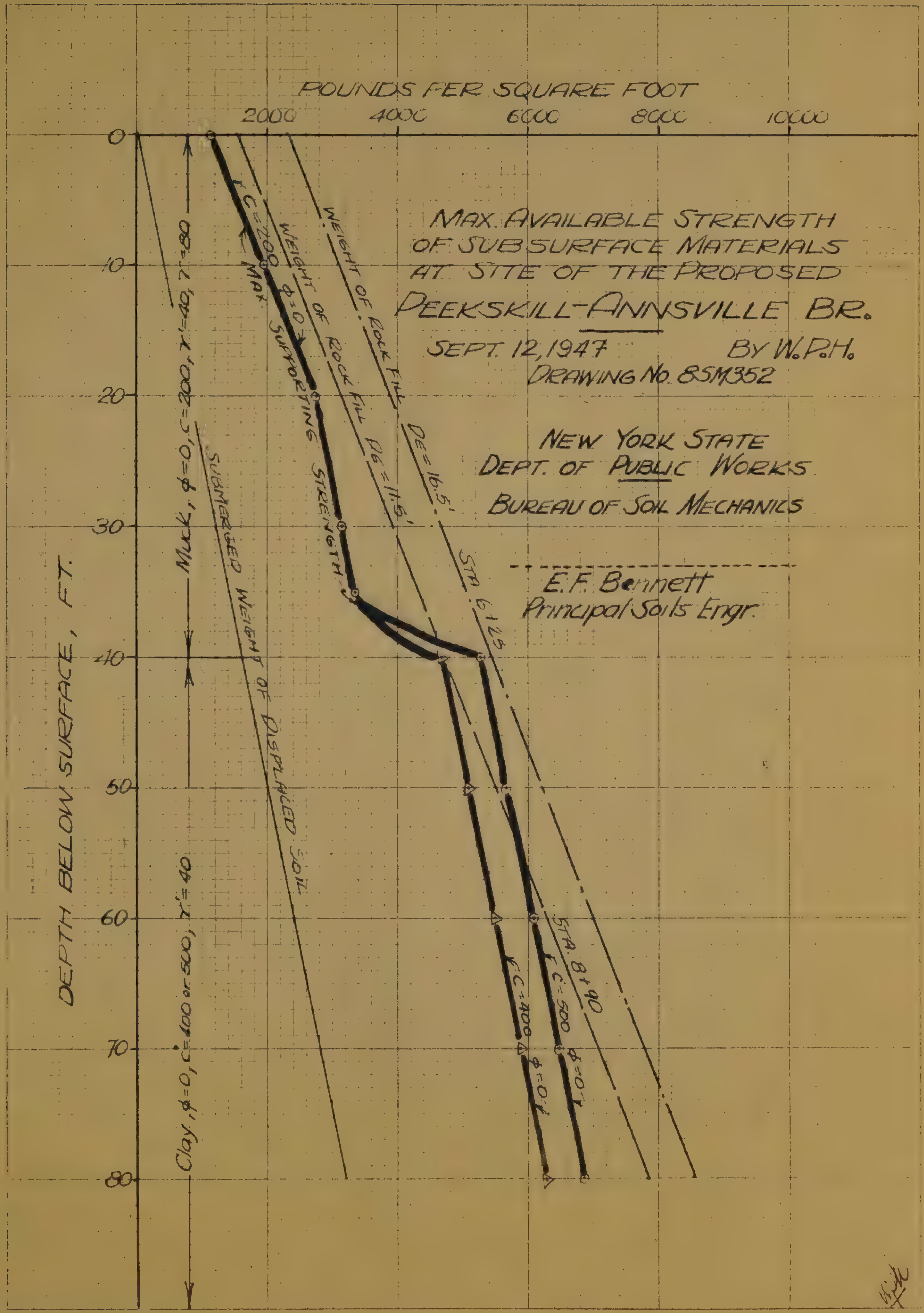
STATE OF NEW YORK
DEPARTMENT OF PUBLIC WORKS
DIVISION OF CONSTRUCTION
BUREAU OF SOIL MECHANICS

PEEKSKILL-ANNSVILLE BR.

COHESION REQUIRED FOR VARIOUS FILL HEIGHTS SF=1

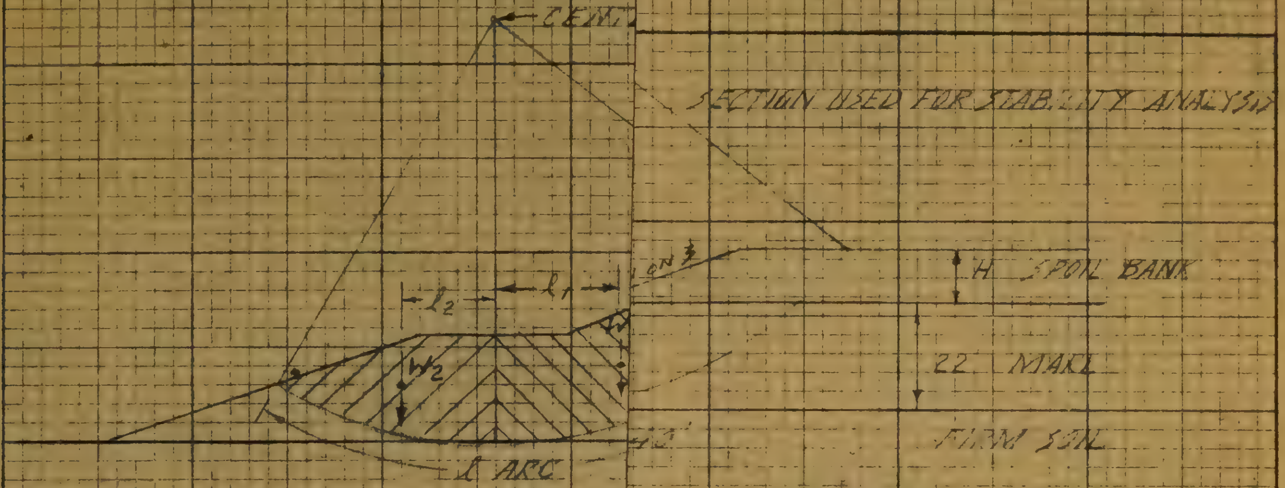
Comp. By W.F. Hofmann
Drawn By W.F. Hofmann

APPROVED	19	DISTRICT NO.	8
EARL F. BENNETT PRINCIPAL SOILS ENGINEER		COUNTY	WEST.
		DRAWING NO.	SSM-42



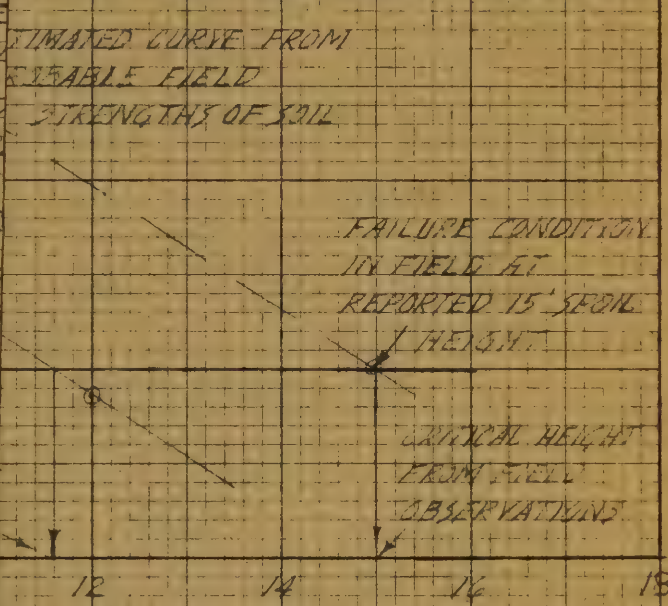
POUNDS PER SQUARE FOOT
2000
4000
6000
8000
10000
0
10
20
30
40
50
60
70
80
DEPTH BELOW SURFACE, FT.
MAX. AVAILABLE STRENGTH
OF SUBSURFACE MATERIALS
AT SITE OF THE PROPOSED
PEEKSKILL-ANNSVILLE BR.
SEPT. 12, 1947
BY W.P.H.
DRAWING No. 8SM352
NEW YORK STATE
DEPT. OF PUBLIC WORKS
BUREAU OF SOIL MECHANICS
E.F. Bennett
Principal Soils Engr.
Muck, φ=0, c=200, γ'=40, γ=80
Clay, φ=0, c=400 or 500, γ'=40
SUBMERGED WEIGHT OF DISPLACED SOIL
MAX. SUPPORTING STRENGTH
f.c=200
φ=0
WEIGHT OF ROCK FILL
WEIGHT OF ROCK FILL DG=11.5'
DE=16.5'
STA. 6+25
STA. 8+90
f.c=400
f.c=500
φ=0

DESCRIPTION OF CIRCULAR ARC (H)
STABILITY ANALYSIS METH



STRENGTH ASSUMPTIONS
MARL = S_f = 220 PSF
SPOIL BANK MARL = S_f = 250 PSF

1. ASSUME FAILURE PLANE IS CIRCU
SURFACE WITH ROTATION ABOU
AS SHOWN. THIS APPROXIMATE
MAJORITY OF FAILURE SURFAC
2. MOMENT OF FORCES CAUSING F
BASED ON WEIGHTS OF SOI
DISTANCES TO CENTER OF
TOTAL FAILURE MOMENT = W_1
3. MOMENT OF RESISTING FORCES
RESISTANCE OF SOIL ALONG A
S_f SHEAR STRENGTH IN LBS
ANALYSIS A SECTION ONE FO
TOTAL RESISTING MOMENT IS
4. FACTOR OF SAFETY FOR STA
MOMENT OF RESISTI
MOMENT OF FAILUR
WHEN FACTOR OF SAFETY IS
WE HAVE A FAILURE CONDIT

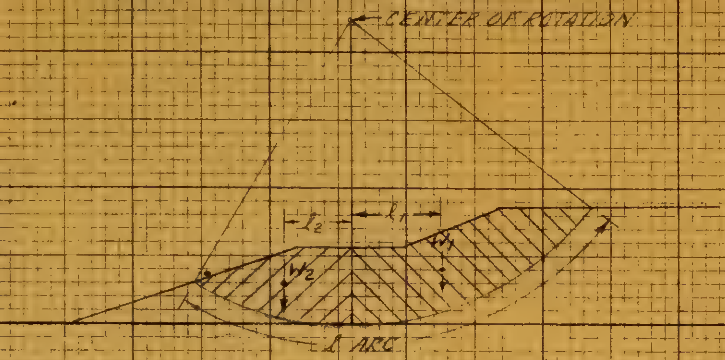


STATE OF NEW YORK
DEPARTMENT OF PUBLIC WORKS
DIVISION OF CONSTRUCTION
BUREAU OF SOIL MECHANICS

STABILITY ANALYSIS
SPOIL BANK SLOPE

APPROVED 19
PRINCIPAL SOILS ENGINEER
DISTRICT NO.
COUNTY
DWG. NO. SM

DESCRIPTION OF CIRCULAR ARC STABILITY ANALYSIS METHOD

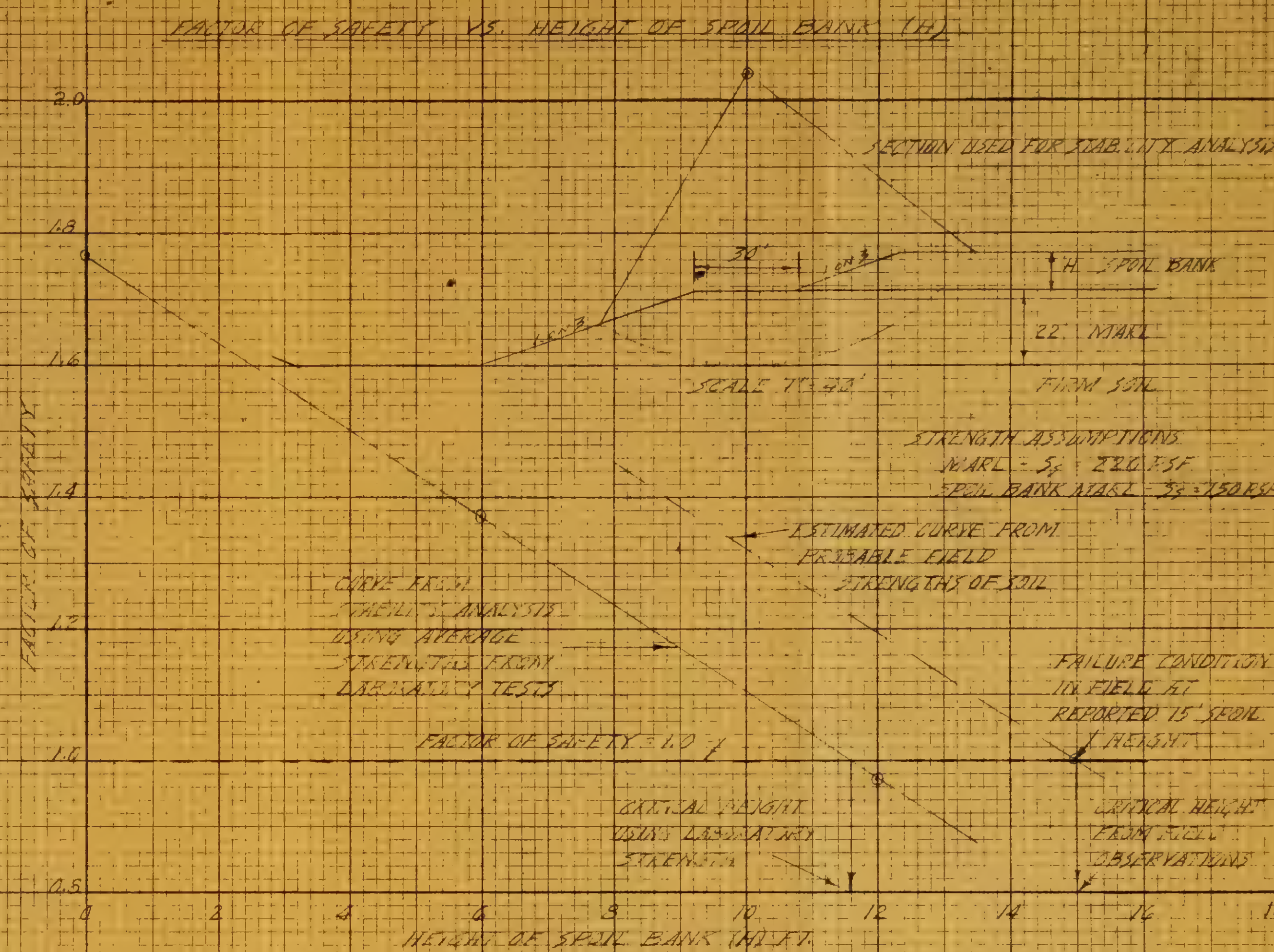


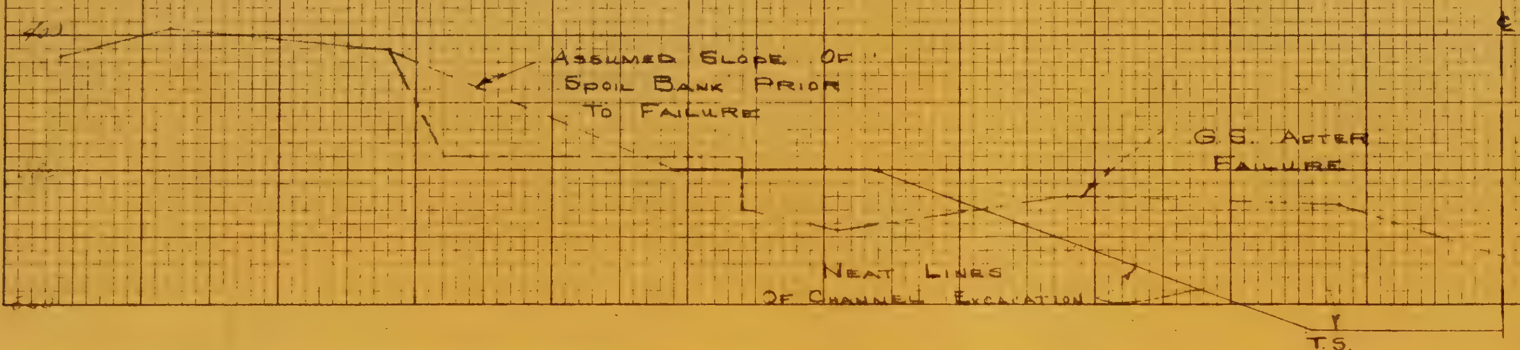
1. ASSUME FAILURE PLANE IS CIRCULAR CYLINDRICAL SURFACE WITH ROTATION ABOUT CENTER OF ARC AS SHOWN. THIS APPROXIMATES SHAPE OF MAJORITY OF FAILURE SURFACES
2. MOMENT OF FORCE CAUSING FAILURE – BASED ON WEIGHTS OF SOIL MASS AND DISTANCES TO CENTER OF GRAVITY
   TOTAL FAILURE MOMENT = $W_1 \ell_1 - W_2 \ell_2$
3. MOMENT OF RESISTING FORCES – BASED ON SHEARING RESISTANCE OF SOIL ALONG ARC.
   S_f – SHEAR STRENGTH IN LBS/SQ. FT. (NOTE – FOR ANALYSIS A SECTION ONE FT. IN DEPTH IS USED)
   TOTAL RESISTING MOMENT IS ℓ_{ARC} TIMES S_f
4. FACTOR OF SAFETY FOR STABILITY =

$$\frac{\text{MOMENT OF RESISTING FORCES}}{\text{MOMENT OF FAILURE FORCES}}$$

WHEN FACTOR OF SAFETY IS LESS THAN ONE WE HAVE A FAILURE CONDITION.

FACTOR OF SAFETY VS. HEIGHT OF SPOIL BANK (H)





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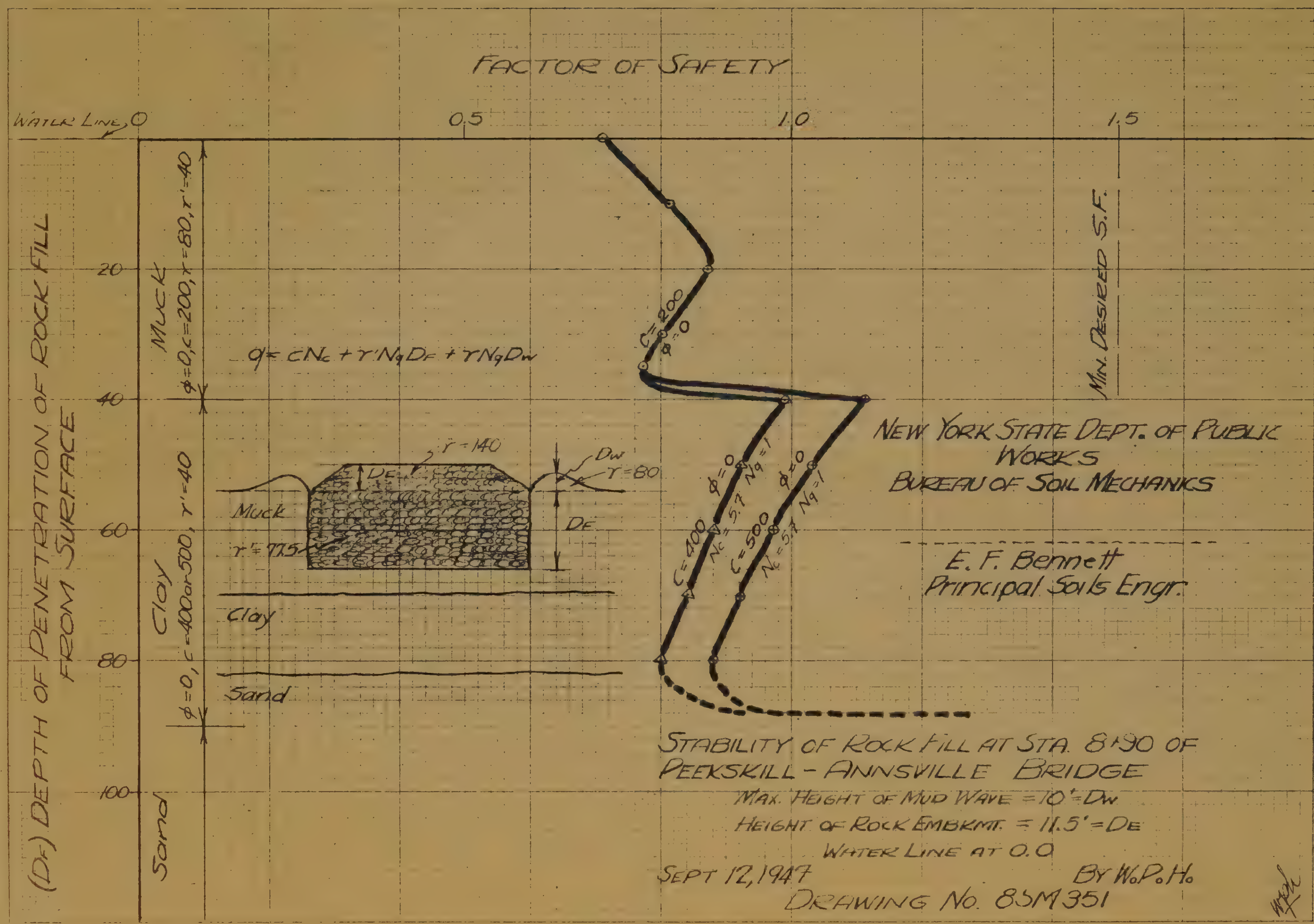
DEPARTMENT OF PUBLIC WORKS
DIVISION OF CONSTRUCTION
BUREAU OF SOIL MECHANICS

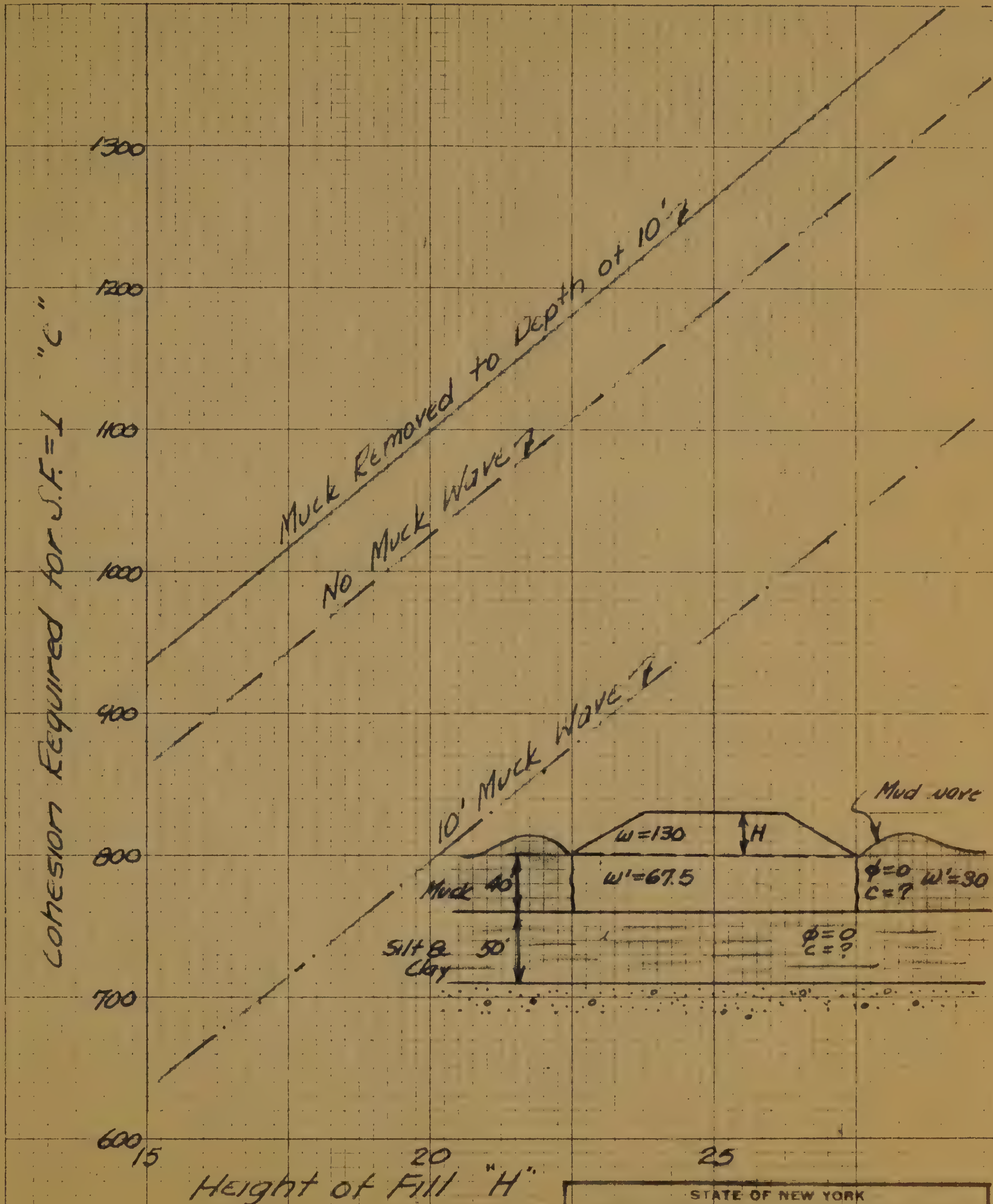
STABILITY ANALYSIS
SPOIL BANK SLIDE
... RIVER RELOCATION

APPROVED ... 19 ...
PRINCIPAL SOILS ENGINEER

DISTRICT NO. ...
COUNTY ...
DWG NO ... SM ...



KEUFFEL & ESSER CO., N. Y. NO. 359T-8G
10 × 10 to the inch.
MADE IN U.S.A.
FACTOR OF SAFETY
WATER LINE 0
0.5
1.0
1.5
MIN. DESIRED S.F.
(DF) DEPTH OF PENETRATION OF ROCK FILL FROM SURFACE
20
40
60
80
100
Muck
ϕ=0, c=200, γ'=80, γ'=40
Clay
ϕ=0, c=400 or 500, γ'=40
Sand
q = cNc + γ'NqDF + γNqDw
c=200
ϕ=0
ϕ=0
Nq=1
c=400
Nc=5.7
ϕ=0
Nq=1
c=500
Nc=5.7
γ=140
DE
Dw
γ=80
Muck
DE
γ'=77.5
Clay
Sand
NEW YORK STATE DEPT. OF PUBLIC WORKS
BUREAU OF SOIL MECHANICS
E. F. Bennett
Principal Soils Engr.
STABILITY OF ROCK FILL AT STA. 8+90 OF
PEEKSKILL - ANNSVILLE BRIDGE
MAX. HEIGHT OF MUD WAVE = 10' = Dw
HEIGHT OF ROCK EMBKMT. = 11.5' = DE
WATER LINE AT 0.0
SEPT 12, 1947
BY W.D.H.
DRAWING No. 8SM351



Comp. By W.F. Hofmann
Drawn By W.F. Hofmann

STATE OF NEW YORK
DEPARTMENT OF PUBLIC WORKS
DIVISION OF CONSTRUCTION
BUREAU OF SOIL MECHANICS

PEEKSKILL-ANNSVILLE BR.

COHESION REQUIRED FOR VARIOUS FILL HEIGHTS SF = 1

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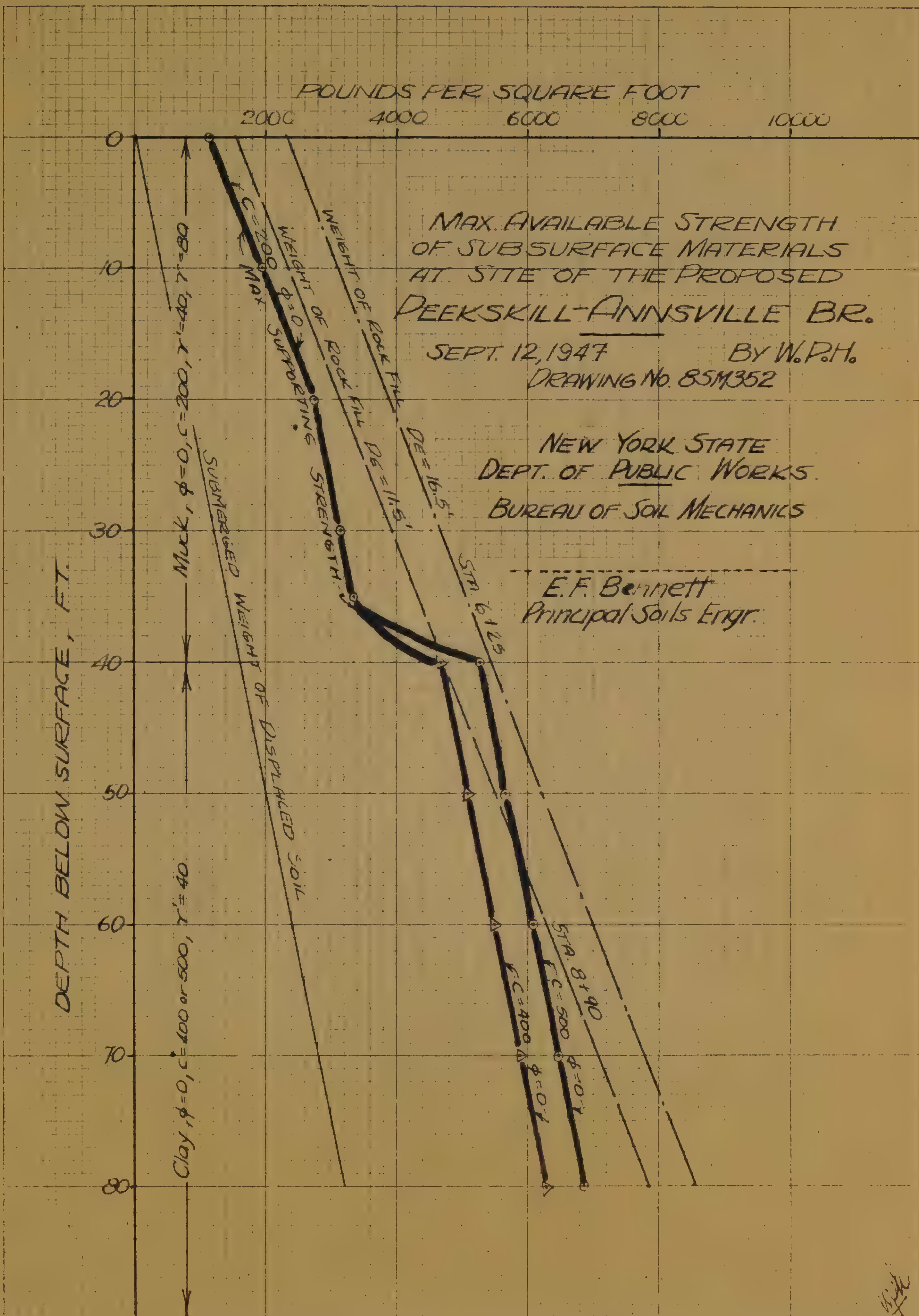
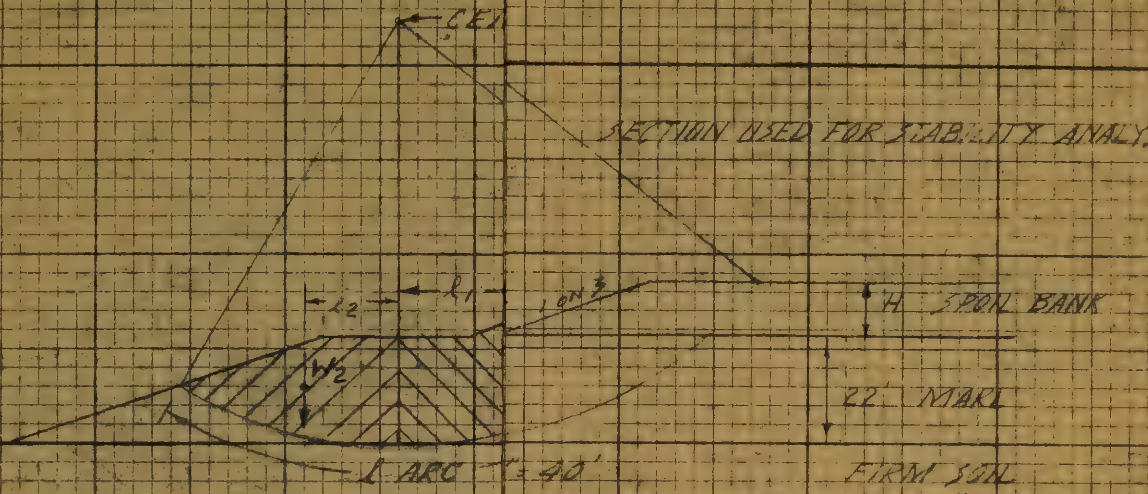
EARL F. BENNETT
PRINCIPAL SOILS ENGINEER

DISTRICT NO. 8
COUNTY WEST.
DRAWING NO. SSM542


POUNDS PER SQUARE FOOT
2000
4000
6000
8000
10000
0
10
20
30
40
50
60
70
80
DEPTH BELOW SURFACE, FT.
MAX. AVAILABLE STRENGTH
OF SUBSURFACE MATERIALS
AT SITE OF THE PROPOSED
PEEKSKILL-ANNSVILLE BR.
SEPT. 12, 1947
BY W.P.H.
DRAWING No. 85M352
NEW YORK STATE
DEPT. OF PUBLIC WORKS
BUREAU OF SOIL MECHANICS
E.F. Bennett
Principal Soils Engr.
Muck, ϕ=0, c=200, γ'=40, γ=80
Clay, ϕ=0, c=400 or 500, γ'=40
SUBMERGED WEIGHT OF DISPLACED SOIL
SUPPORTING STRENGTH
WEIGHT OF ROCK FILL Dб=11.5'
WEIGHT OF ROCK FILL Dб=16.5'
STA. 6+25
STA. 8+90
c=200 ϕ=0
c Max.
c=400 ϕ=0
c=500 ϕ=0

DESCRIPTION OF CIRCUL

STABILITY ANALYSIS MEBANK (H)



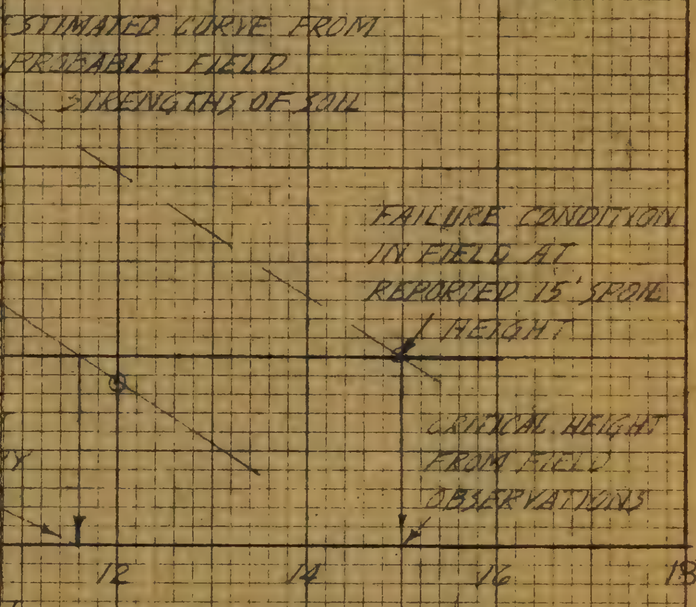
1. ASSUME FAILURE PLANE IS CIRC
SURFACE WITH ROTATION ABO
AS SHOWN. THIS APPROXIMAT
MAJORITY OF FAILURE SURFA

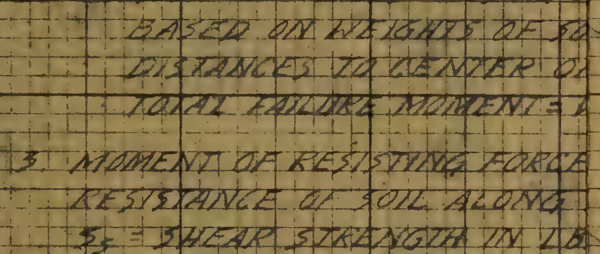
2. MOMENT OF FORCES CAUSING
BASED ON WEIGHTS OF SO
DISTANCES TO CENTER O
TOTAL FAILURE MOMENT = V

3. MOMENT OF RESISTING FORCE
RESISTANCE OF SOIL ALONG
S_s = SHEAR STRENGTH IN LB
ANALYSIS A SECTION ONE F
TOTAL RESISTING MOMENT I

4. FACTOR OF SAFETY FOR ST
MOMENT OF RESIST
MOMENT OF FAILUR
WHEN FACTOR OF SAFETY
WE HAVE A FAILURE CONDI

STRENGTH ASSUMPTIONS
MARL - S_s = 220 PSF
SPOIL BANK MARL - S_s = 750 PSF





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STABILITY ANALYSIS
SPOIL BANK SLIDE
CLYDE RIVER RELOCATION
TYRE - MONTEZUMA CT SX-2

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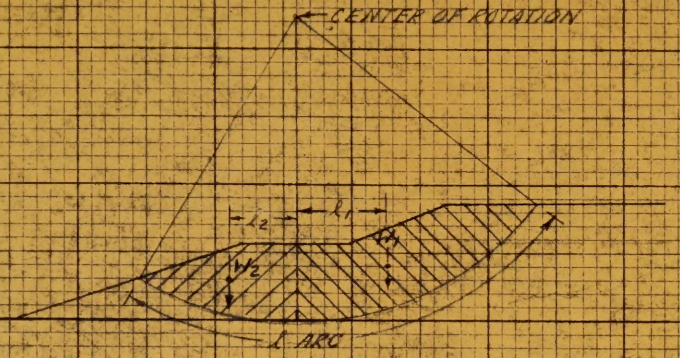
PRINCIPAL SOILS ENGINEER

DISTRICT NO.
COUNTY
DWG. NO 3 SM

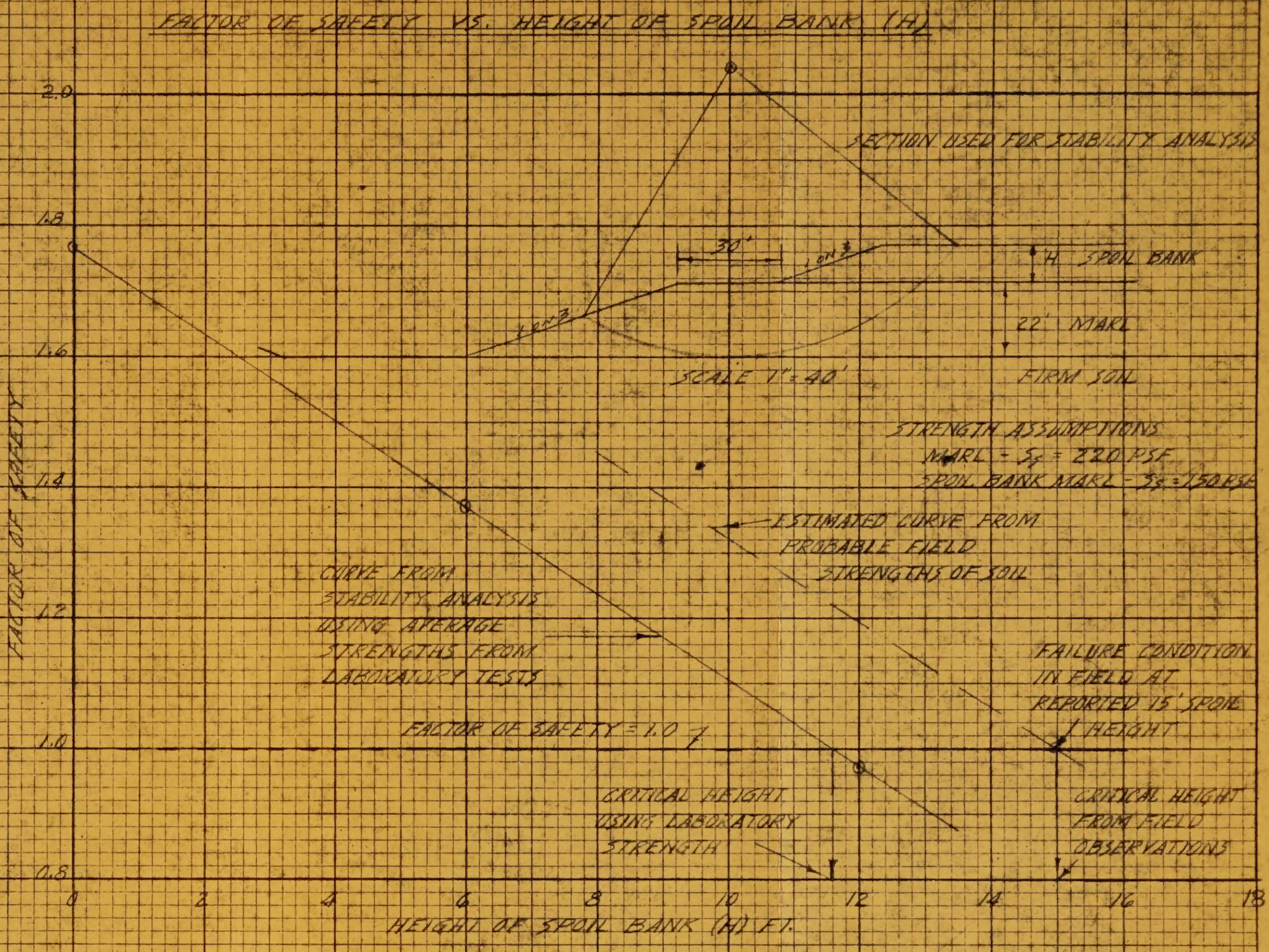
DESCRIPTION OF CIRCULAR ARC STABILITY ANALYSIS METHOD

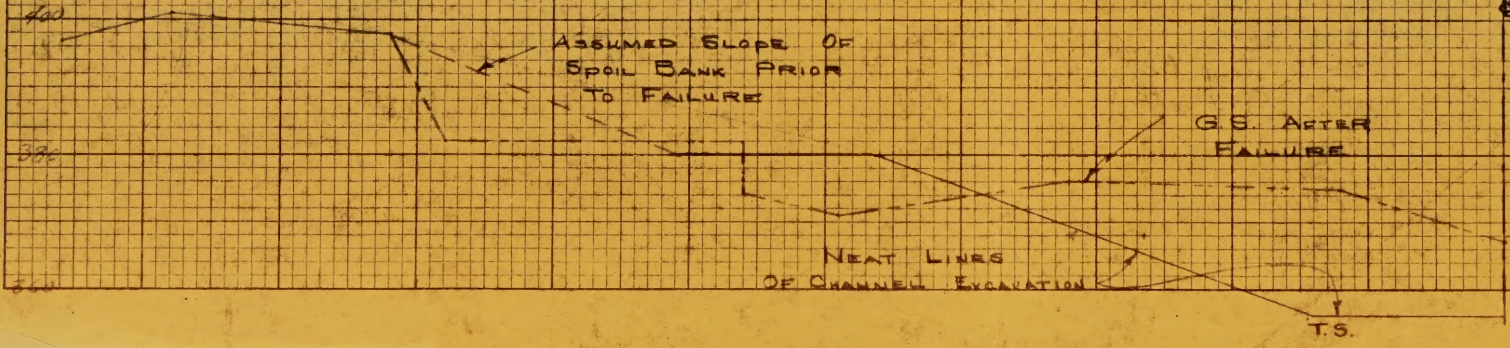


1. ASSUME FAILURE PLANE IS CIRCULAR CYLINDRICAL SURFACE WITH ROTATION ABOUT CENTER OF ARC AS SHOWN. THIS APPROXIMATES SHAPE OF MAJORITY OF FAILURE SURFACES.
2. MOMENT OF FORCES CAUSING FAILURE – BASED ON WEIGHTS OF SOIL MASS AND DISTANCES TO CENTER OF GRAVITY. TOTAL FAILURE MOMENT = $W_1 l_1 - W_2 l_2$
3. MOMENT OF RESISTING FORCES – BASED ON SHEARING RESISTANCE OF SOIL ALONG ARC. S_f = SHEAR STRENGTH IN LBS/SQ. FT. (NOTE – FOR ANALYSIS A SECTION ONE FT. IN DEPTH IS USED) TOTAL RESISTING MOMENT IS l_{ARC} TIMES S_f
4. FACTOR OF SAFETY FOR STABILITY = $\frac{\text{MOMENT OF RESISTING FORCES}}{\text{MOMENT OF FAILURE FORCES}}$

WHEN FACTOR OF SAFETY IS LESS THAN ONE WE HAVE A FAILURE CONDITION.

FACTOR OF SAFETY VS. HEIGHT OF SPOIL BANK (H)





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STABILITY ANALYSIS
SPOIL BANK SLIDE
CLYDE RIVER RELOCATION
TYRE-MONTEZUMA CT 5X-6

APPROVED 19	DISTRICT NO. 3
	COUNTY SENECA
PRINCIPAL SOILS ENGINEER	DWG. NO. 3 SM 1568